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"Modern Methods of Structural Design."

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INDIVIDUAL experience and particular conditions play so important a part in the complete process of design that it would be unprofitable, even if it were possible here, to attempt a detailed consideration of all that design means. The scope of this Lecture will be strictly limited to that part concerned with the strength of the structure.

Even in this comparatively narrow field, the designer's task of producing a structure of adequate strength is complex. So complex is it that assumptions must be made, not only to make good deficiencies in the knowledge of conditions but with the avowed object of simplifying the strength-calculations. Whatever advances in knowledge may be made, it is unlikely that this state of affairs will change radically ; those of you engaged in design know that involved methods of calculation are usually impracticable. Simplification can, however, be bought at too great a price. There is, in some branches of structural engineering, a tendency for assumptions to be chosen because they make some simple calculation possible, even though they are so sweeping that the true behaviour of the structure is disguised. Under these conditions, while it may be possible, with the safeguard of large load-factors, to produce stable structures, any evolution of the method of construction is impossible. Although regrettable, this is not surprising. The designer is not averse to adopting improved methods when they are found, but he cannot,

[1] This Lecture was delivered at a meeting of the Association of London Students, and was repeated before the Local Associations at Belfast, Birmingham, Bristol, Cardiff, Glasgow, Manchester, Newcastle, and Sheffield.—ACTING SEC. INST. C.E.

except when dealing with a major structure, such as a Sydney Harbour bridge, afford to devote his time to the search. Other investigators, though giving a great deal of attention to the development of new methods of stress-analysis, have done relatively little to produce new methods of design. Developments in stress-analysis do not influence design directly, since they do no more than make easier the determination of the stress-distribution in a structure when the sections of the members have been chosen, and it is only in the more important cases that the labour of making a tentative design and modifying it by subsequent analysis is offset, economically, by the greater efficiency of the final result. That the production of design-methods has lagged behind stress-analysis is shown by the fact that, apart from the beam resting on simple supports, the only form of structure that can be designed directly by methods common in practice, with any confidence that the maximum stresses in the members produced by the assumed loading will be a close approximation to the permissible stresses chosen, is the simple truss having pin-joints. Even there, owing to the behaviour of bars subjected to compressive loads and to the somewhat archaic formulas still commonly used for the permissible end-loads on such bars, it is probable that the maximum stress in a strut will not be a close approximation to the value intended by the designer. Although the simple truss is a common form, it is rarely built with effective pin-joints. More often in practice, by the use of riveting to gusset-plates, the joint is practically rigid, preventing appreciable relative rotation of the ends of the members joined and thus introducing bending restraints when load is applied. These restraints develop bending stresses in the members, and in all but the lightest trusses it is necessary to see that they are not excessive. Since the magnitudes of these secondary bending stresses can only be gauged when the proportions of the members are known, this is a case where tentative design based on a simplifying assumption is used. It is, therefore, also one which is more readily affected by advances in stress-analysis, and it may be worth while to draw attention to a new approach to the problem which has been made possible by the recent work of Hardy Cross, Professor of Structural Enginering in the University of Illinois. His original Paper [1] should be studied by all students for two reasons. It sets out a novel principle, having already far-reaching effects on the designer's attitude towards those problems of stress-analysis arising in rigidly-jointed structures which the use of reinforced concrete and of welded steelwork have brought to the fore. It does so in a refreshingly concise and simple form not usual

[1] H. Cross, "Analysis of Continuous Frames by Distributing Fixed-End Moments," Trans. Am. Soc. C.E., Vol. 96 (1932), p. 1.

in technical papers and extremely difficult to emulate successfully. As the Cross or moment-distribution method may not be familiar to all of you it will be well here, before considering the effect of the bending stress on the strength of a truss, to give an example of the determination of this stress by the new method. It should, however, first be noted that, whilst the rigidity of joints may in some cases have an appreciable effect on the deflexions of a truss, it is usual to be satisfied with a first approximation and to assume that the deflexions of the joints of the actual truss are the same as those of the frame with pinned joints in place of rigid joints. These deflexions are easily calculated, and the problem of secondary-stress determination is, therefore, that of finding the bending stresses in a rigidly jointed truss when the joints are forced to take up known deflexions.

In the classic slope–deflexion method which will be familiar to you all, the deflected structure is first considered to be pin-jointed, with the result that the angles between the bars change. The moments which must be applied to the members to force their ends back into the relative angular positions set by the rigid joints are then determined by forming and solving as many simultaneous equations as there are joints in the frame. In the Cross method, on the other hand, the joints are kept rigid throughout. They are first forced to take up the pre-determined deflexions, but any rotation of the joints is prevented. The resulting end moments can be written down without trouble but they are not, of course, the final values giving rise to the secondary stresses, since it is necessary to apply external couples at each joint to prevent rotation. These external couples must be removed; each joint is released in turn while the others are kept fixed. A joint, on the removal of the external couple, rotates until it reaches an equilibrium position, and in doing so modifies the moments in the members connected to it. These modifications can be found without difficulty, and the process of releasing joints is continued until the changes in the end moments become small enough to be neglected.

The method will be more readily understood by reference to an example. The secondary bending stresses produced in the cantilever frame shown in *Fig. 1* * when a load of 20,000 lbs. is hung from R will be determined. All the members of the frame have the same cross-section and are connected together through gusset-plates which prevent any change in angle between the ends of the members. When the load is applied the structure deflects, joints Q and R moving down the same distance, 0·038 inch, and joint R moving to the left 0·010 inch, deflexions which can be found with ease by means

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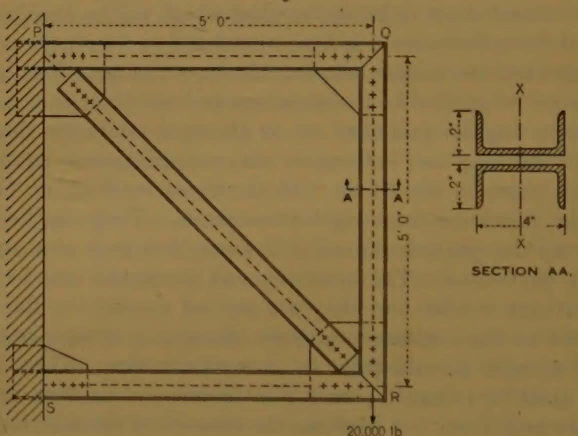
of a Williot diagram if the assumption of pin-joints, already referred to, is made. The condition of the rigidly-jointed structure can be gauged to some extent from the behaviour of the model, built of spring steel and attached at P and S to a back-board, illustrated in *Figs. 2*. It is shown unloaded in *Figs. 2* (*a*) and in its final deflected form under load in *Figs. 2* (*d*), where the bending induced in the members is apparent.[1]

The model can be used conveniently to show the procedure in the Cross method of determining the secondary bending stresses. As I have said, in the first step when the joints are moved into their predetermined deflected positions all rotation is prevented. This step can be carried out on the model by moving joints Q and R through

Fig. 1.

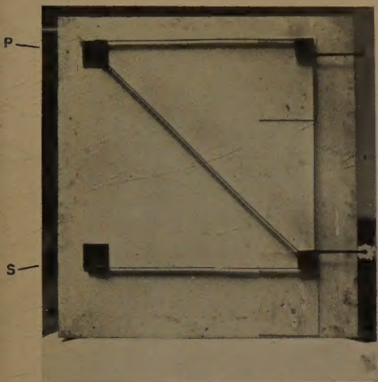


Young's modulus$=30\times10^6$ lbs. per square inch.
Relevant moment of inertia of member (I_{xx})$=10$ inch^4 units.
Cross-sectional area of member$=4$ square inches.

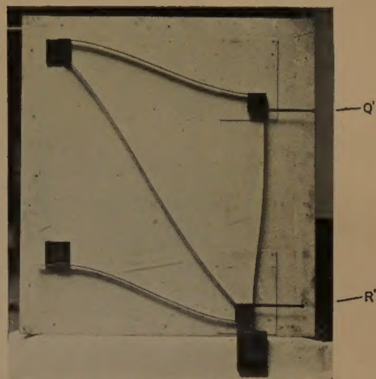
the distances already calculated and inserting stops between the gusset-plates and the back-board so that the joints retain their original angular positions. The state of the model at this stage is shown in *Figs. 2* (*b*), and it is clear from the positions of the pointers Q′ and R′, which are rigidly attached to the gusset-plates, that whilst deflexion of the joints has been allowed, rotation has been prevented. Each member is, therefore, behaving as an encastré beam and, due to the relative deflexion of the ends, fixing moments of magnitude $\frac{6EI\delta}{L^2}$ are induced at the ends, E denoting Young's

[1] In order to make the demonstration clear, exaggerated deflexions of the model, *Figs. 2*, were obtained by making the ends of the diagonal at P and the strut at S free to slide longitudinally.

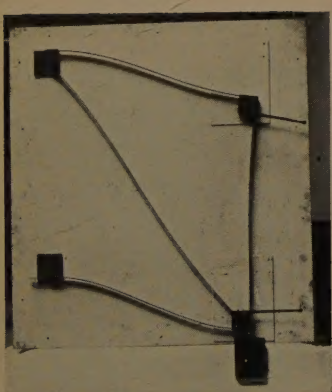
Figs. 2.



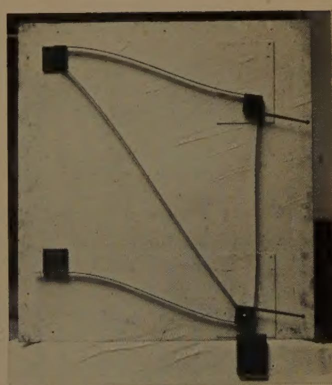
(a)



(b)



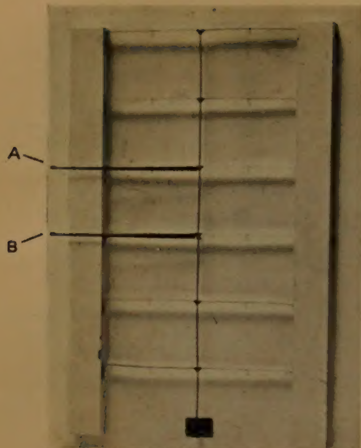
(c)



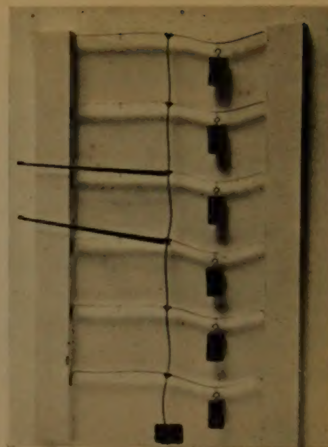
(d)

DEFORMATION OF CANTILEVER FRAME.

Figs. 9.



(a)



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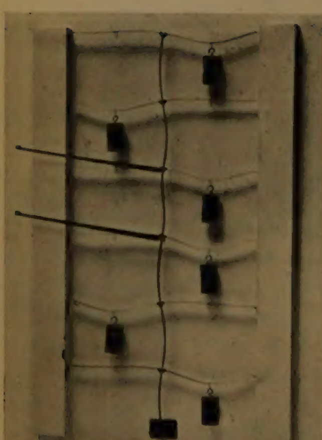
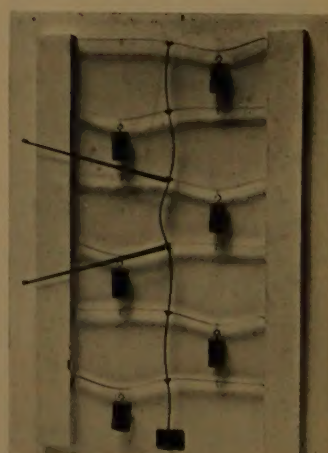


(c)



(d)

FRAME WITH LOADS ARRANGED TO PRODUCE DOUBLE- AND SINGLE-CURVATURE BENDING.

modulus, I the moment of inertia or second moment of area of cross-section, δ the relative deflexion of the ends, and L the length of the member. These moments are set down in the calculation, and it is convenient to arrange the work in tabular form, giving one column to each end of each member as in Table I. At the ends of the members PQ and RS the fixing moments are 19,140 lbs.-inches and at the ends of QR and RS they are 5,000 and 8,535 lbs.-inches respectively (Table I). In this particular case these moments, applied to the ends

TABLE I.

Joint P

(1)	(2)
— 8535	— 19140
0	+ 6035
+ 3465	0
0	— 1232

Joint Q

(3)	(4)	
— 19140	— 5000	(a)
+ 12070	+ 12070	(b)
0	+ 4928	(c)
— 2464	— 2464	

Joint S

(5)
— 19140
0
+ 4928
0

Joint R

(6)	(7)	(8)	
— 19140	— 8535	— 5000	(a)
0	0	+ 6035	(b)
+ 9855	+ 6930	+ 9855	(c)
0	0	— 1232	

of the members by the joints, are all counter-clockwise, so that each is entered in the Table with a negative sign. To prevent rotation of a joint an external couple has been applied to it through the stops ; its magnitude is known since it must be equal to the sum of the fixing moments acting on the ends of the member at the joint ; these are already set down Thus at joint Q the external couple is (— 19,140 — 5,000) or — 24,140 lbs.-inches. Before the true values of the secondary end-moments can be determined the joints must be free ; that is to say, the external couples must be removed. To

make the calculation simple the external couples are not all removed at the same time. First release Q, which, in the model, means removing the stops inserted between it and the back-board, keeping the other joints fixed. Q rotates into a new equilibrium position, as can be seen from *Figs. 2* (*c*), and in so doing modifies the end moments on the members. The modification can be calculated with ease since the removal of an external couple of — 24,140 lbs.-inches induces the same moments in the members as the application of a couple + 24,140 lbs.-inches. In the frame under consideration the stiffness (the moment of inertia divided by the length) of each of the members meeting at Q is the same, so that the moments developed at the ends Q of the members by the rotation of Q will be of the same magnitude, one half of + 24,140, or + 12,070 lbs.-inches [line (*b*), columns (3) and (4), Table I], since the moment induced is in proportion to the stiffness of the member. The rotation of Q also modifies the moments at the far ends of these members QP and QR, producing additional moments there of + 6,035 lbs.-inches [line (*b*), cols. (2) and (8)]. This is so because the moment, induced at the fixed end of a beam by a moment M at the other end, causing rotation only, is $\frac{1}{2}M$. Since joint R has not been released the moments at the ends of the other members remain unchanged [line (*b*), cols. (1), (5), (6) and (7)]. Joint Q is now in equilibrium. It is fixed once more in its new position, by the insertion of stops, so that it cannot rotate. The next joint R is now released. The external moment acting on this joint is the algebraic sum of all those so far set down, in cols. (6), (7) and (8), and is — 26,640. On release the joint rotates into its new equilibrium position, with the result that this moment is divided between the members in proportion to their stiffnesses [line (*c*), cols. (6), (7) and (8)] and moments of half these magnitudes are induced at the far ends of the members [line (*c*), cols. (1), (4) and (5)]. Joint R is once more fixed. When we consider the joint Q again we find that an external couple has again been applied to it through the stops to prevent it rotating, since a moment + 4,928 was carried over as a result of the rotation of R. When the joint is released, therefore, it once more rotates and modifies the moments further and so the process is continued. For practical purposes, since the modifications produced by subsequent releases are clearly small, the calculation need be carried no further than this and the bending moment at the end of each member is found by adding up the moments in the respective columns. Thus, at the end Q of PQ the moment is — 9534 lbs.-inches. From these end moments the secondary bending stresses are calculated.

It is usual, when checking to see that the structure is not overstressed, to add the maximum secondary end bending stress arising

from the end moments calculated in this way to the primary or axial stress in each member determined from a consideration of the pin-jointed truss, and to see that this total end stress does not exceed that permissible for the material. This permissible stress is, in the case of a compression member, usually taken to be some multiple (for instance $1\frac{1}{3}$ times) of the safe axial load per unit area on a pin-ended strut, that is, on a strut having no end moments applied. The members of a rigidly-jointed truss are, however, subjected to axial load and to end moments also. To disguise this fact when deducing the permissible stress frequently means that the best use is not made of the material. It is probable that the majority of compression-members in a truss are bent in double curvature (for example, member RS, *Figs. 2* (*d*)), the end moments having the same sense, and as will be seen later the maximum end stress which can be allowed in such a member is considerably greater than the safe axial load on the pin-ended strut. On the other hand, it is possible to find compression members in trusses bent in single curvature by the secondary moments, the moment at one end tending to cause a clockwise rotation and that at the other a counter-clockwise rotation, and there the permissible end stress is less than that usually adopted.

When a member AB of length L is subjected to an axial compressive force P and end moments M_A and M_B, the bending moment at any section, distance x from one end, is given by the expression

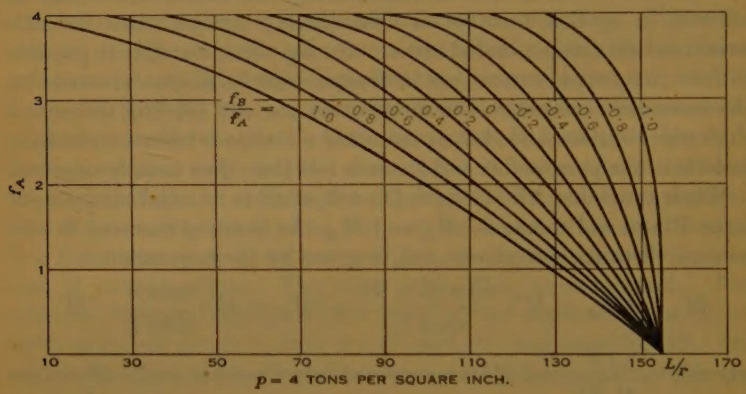
$$M_x = (M_A + M')\frac{\sin\alpha\,(L-x)}{\sin\alpha\,L} + (M_B + M')\frac{\sin\alpha\,x}{\sin\alpha\,L} - M'$$

where $\alpha = \sqrt{\frac{P}{EI}}$, and M' is a constant introduced to make allowance for the imperfections in a practical member, which we know, from the examination of test-results, can be conveniently represented by a small initial curvature of the axis.

From this equation the maximum stress in the member can be determined, and although the expression is too cumbersome for direct use in design it is not a difficult matter to translate it into the form of curves giving for any value of axial load per unit area and any ratio of the end bending moments M_B and M_A the maximum end bending stress f_A which can be induced with safety at the end of the member. Such a family of curves, based on a total maximum stress in the member of 8 tons per square inch, is shown in *Fig. 3* for an axial load per unit area (p) of 4 tons per square inch. Each curve refers to a different ratio of end bending moments, and you will see that when the member is bent in double curvature by end moments of equal magnitude $\left(\frac{M_B}{M_A} = \frac{f_B}{f_A} = -1{\cdot}0\right)$, the case covered

by the right-hand curve, a secondary end bending stress of 4 tons per square inch would be permissible in any member having a slenderness-ratio (of length to least radius of gyration, L/r) of less than 128; that is to say, the total stress at the end of the member could be the full 8 tons per square inch, a very much greater value than that given by the usual method. The reason is, of course, that in any compression-member having a slenderness-ratio less than 128 subjected to an axial load per unit area of 4 tons per square inch and to equal double-curvature moments, the maximum bending stress will be found to occur at the end. For values of slenderness-ratio greater than 128 the maximum bending stress is found at some intermediate section, and since the total maximum stress made up

Fig. 3.



of axial load per unit area (4 tons per square inch) plus maximum bending stress is limited to 8 tons per square inch, the end bending stress must be less than 4 tons per square inch.

For the case of pure single-curvature bending, represented by the left-hand curve, when the member is bent by equal end moments of opposite sense $\left(\frac{M_B}{M_A} = \frac{f_B}{f_A} = 1{\cdot}0\right)$, the permissible secondary stress is very much smaller, being no more than 1 ton per square inch when the slenderness-ratio is 128 and only 3·7 tons per square inch for a slenderness-ratio of as low as 30.

Although attention has been drawn to these facts in the past, they do not appear to have been incorporated in any design method. The reason in all probability is that, quite apart from the natural objection which a designer would have to the use of a number of families of curves in place of his simple strut formula, they can only be used when the most rigorous stress-conditions for each member are known.

These conditions may not be brought about by that arrangement of load which produces the maximum axial load in the member, since, as the curves show, the total stress at any section depends not only on the axial load and on the maximum end bending stress but also on the ratio of the end bending stresses. A great deal of preparatory work would therefore have to be carried out to determine worst conditions in any particular type of truss before design could be started, and as it would be unsound to base a new method on theoretical argument alone, without the support of experimental data collected on actual trusses, the complete investigation called for would be a heavy piece of work. Such an investigation has been carried through recently for another type of structure, the steel building-frame, and a short description of it will show how more exact design methods can be developed. The investigation to which I refer is that made by the Steel Structures Research Committee between the years 1929 and 1936, a full account of which is contained in the Reports of that Committee published by H.M. Stationery Office in 1931, 1934 and 1936. Although directed primarily to the steel frame, a large part of this investigation dealt with problems common to reinforced-concrete structures. It must not be thought, therefore, that an important type of structure is being entirely neglected in this Lecture, or that all is well with reinforced-concrete design while much has had to be learnt about steel.

While the end bending effects which we have already discussed are of secondary importance in many trusses, there are certain types of structures, including building-frames, which depend upon them for their stability. A review of the methods of designing steel building-frames, which consist in the main of rows of vertical stanchions to which are connected lines of horizontal beams, has shown, however, that it is usual to assume, when considering the effect of vertical loads, that the beams are connected to the stanchions by perfect hinges which supply no bending restraints; in effect the stress-analysis is confined to what may be termed the primary system. This assumption makes the design of the beams a simple matter and has much to recommend it in spite of its extravagance. Its effect on stanchion-design is not so satisfactory, as those of you know who have been engaged on this type of structure. Although on the face of it the choice of the requisite stanchion-section is simplified thereby, the nature of the resulting stress-distribution is so distorted as to make a rational choice impossible.

When the effect of lateral or wind loads is considered it is easy to see that the assumption of hinged joints would result in the deduction of high bending stresses in the stanchions, since each stanchion would behave as a cantilever beam, of length equal to the height of the

building, subjected to a share of the wind loads. The designer, knowing that these excessive stresses do not exist, assumes, when considering lateral loads, that the joints between the members are completely rigid. Thus the assumptions made in designing a frame are diametrically opposite and depend on the type of loading under consideration.

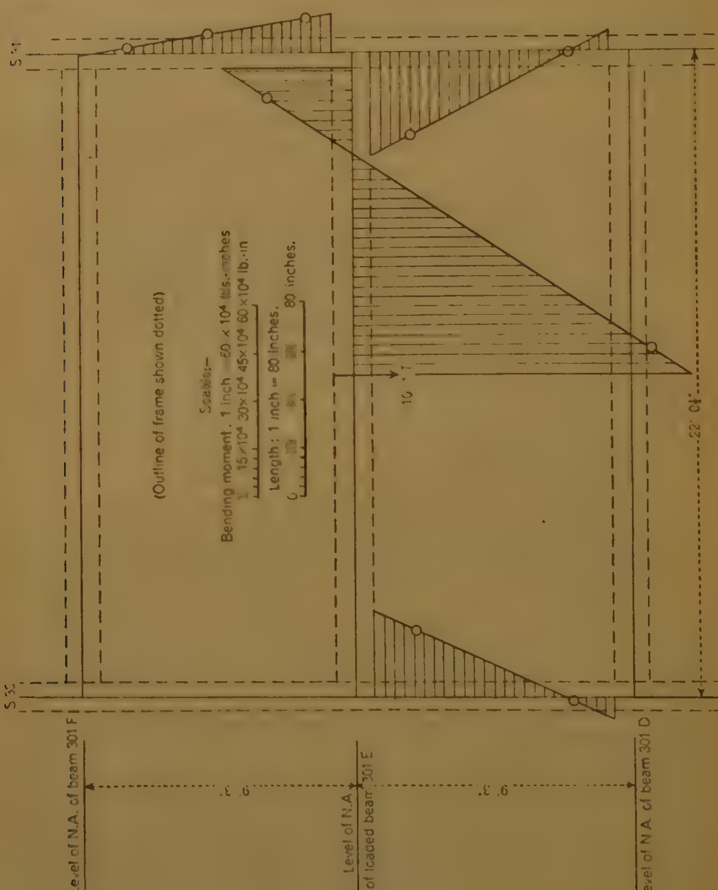
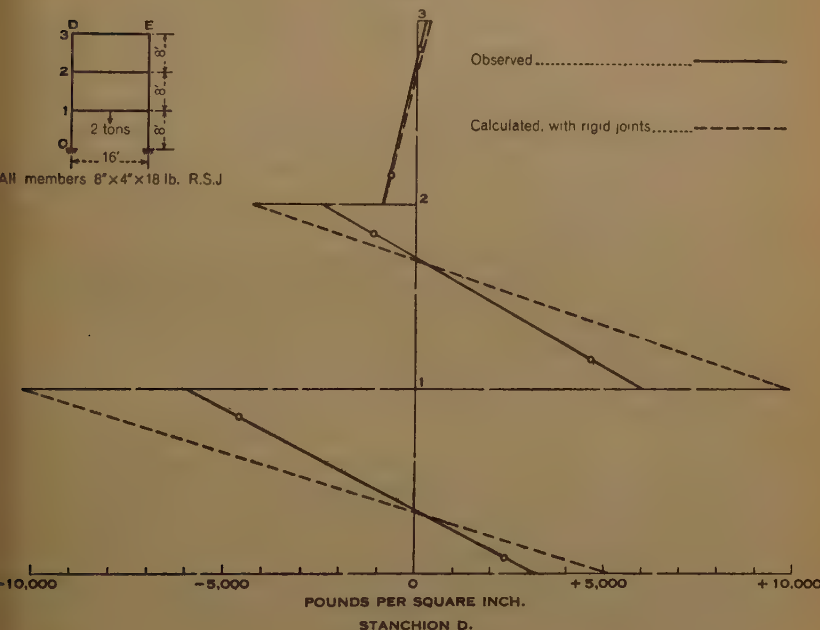


Fig. 4.

A comprehensive series of tests recently made for the Steel Structures Research Committee on a number of existing buildings has shown that the assumption of rigid joints is much more nearly justified than that of pin-joints, whatever the loading. This can be seen from *Fig. 4*, which shows the observed bending moments in part of a single-bay frame of a London hotel-building due to the

application of a concentrated load to the centre of a beam. You will notice first the large restraining moment at the right-hand end of the beam which reduces the maximum stress in that member below the value it would have when simply supported and subjected to the same load. The magnitude of this restraining moment was 470,000 lbs.-inches due to the 10·1-ton load on the beam, and it induced in the stanchion-lengths above and below the beam moments of 120,000 and 350,000 lbs.-inches, values much greater than those

Figs. 5.



which would be assumed in normal design. It is worth pointing out here that the distance from the centre-line of the stanchion at which a reaction of 5·05 tons, half the applied load, would have to act to produce these moments is approximately 42 inches. The connexion responsible for this "equivalent eccentricity" consisted of a bottom stool, a 4-inch by 4-inch by $\frac{1}{2}$-inch top flange-cleat 8 inches long, and two 4-inch by 4-inch by $\frac{1}{2}$-inch web-cleats 10 inches long to a 14-inch deep beam.

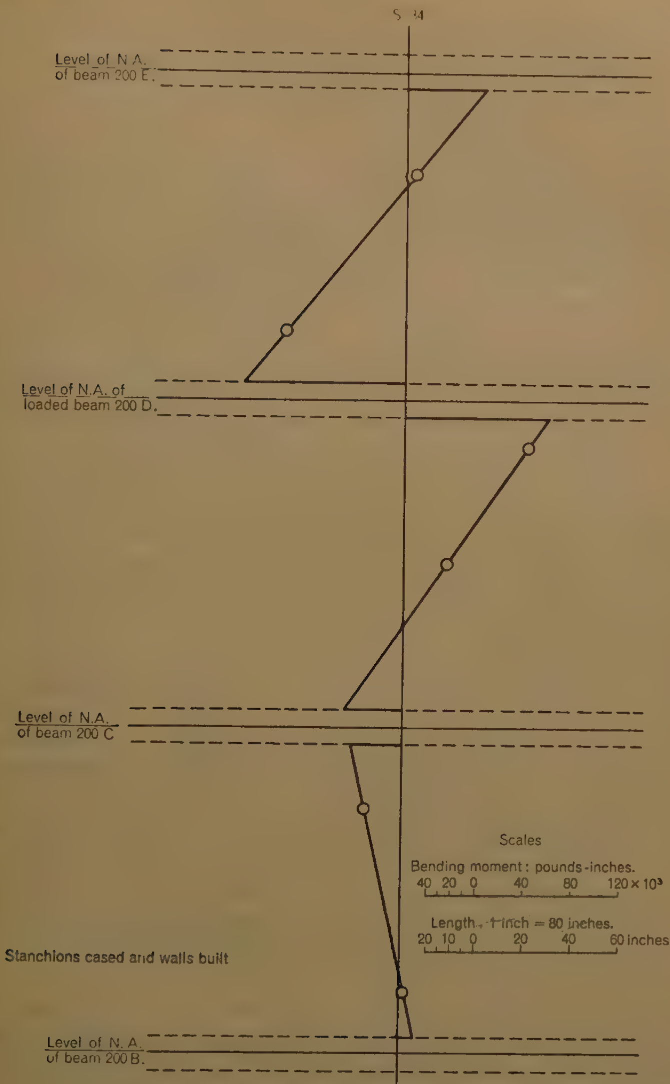
The distribution of bending moment in the stanchions can also be seen, and it is well illustrated in *Figs. 5*, which shows the observed bending stresses in three lengths of a single-bay frame, made up of 8-inch by 4-inch by 18-lb. joists throughout, due to the application of

a central load of 2 tons to the bottom beam. The connexions consisted of the usual type of bottom and top flange-cleats made up of $3\frac{1}{2}$-inch by 3-inch by $\frac{5}{16}$-inch angle 4 inches long, secured with $\frac{1}{2}$-inch diameter bolts to the webs of the stanchions. The observed bending stresses which are shown by the full line were appreciable, the maximum value being 6,075 lbs. per square inch, and you will see by comparing these stresses with those calculated on the assumption of rigid joints, shown by the hatched line, that, although the observed stresses were rather less than those calculated, the form of the observed diagram was the same as that for the rigid-joint case. It is interesting to note that appreciable bending stresses were developed in the top stanchion-length and that since no sway, or horizontal deflexion of the beams, took place the bending of each stanchion-length was in double curvature. This distribution of stanchion-stress is typical of that found in the other buildings tested, and it was not modified in form when the steel frame was clothed with floors, walls and stanchion-casing, as can be seen from *Fig. 6*, which shows the bending moments for three lengths of a stanchion in a London office-building due to a distributed load applied to one-half of the floor at level D when the building was complete with external walls and brickwork stanchion-casing. Here, again, the bending is in double curvature, and the appreciable bending moment in the bottom length remote from the load is again apparent.

As I have said, the bending stresses induced in the stanchions were found to be much greater than those assumed in design to-day, which are taken to be due to the application of a moment equal to the reaction at the end of the loaded beam multiplied by the eccentricity of the reaction, that is the distance of the point of support of the end of the beam from the centre-line of the stanchion. The difference can be gauged from the fact that in the bare framework of the hotel-building the eccentricity at which the reaction from one beam framing into the web of a stanchion would have had to act to produce the bending stresses observed was 44·6 inches, whereas the distance taken in design to-day might have been as low as $\frac{1}{4}$ inch and would certainly not have been more than $2\frac{1}{4}$ inches. The restraining moment developed at the end of the beam in this particular case reduced the maximum stress in that member by 25 per cent. of the value it would have had in the similar simply-supported beam, the condition assumed in design to-day.

The placing of floors and fire-resisting casing, an essential part of a large class of steel-frame buildings, whilst it does not modify the form of the stress-distribution in the steelwork, apart from tending to eliminate the stresses due to sway, has the effect of increasing the rigidity of the connexions and therefore, other things being

Fig. 6.



unchanged, the moments transmitted through those connexions. In one part of the office-building which was tested this increase was found to be as much as 3 times.

The observations showed quite conclusively, therefore, that all the standard types of bolted and riveted beam-to-stanchion connexions

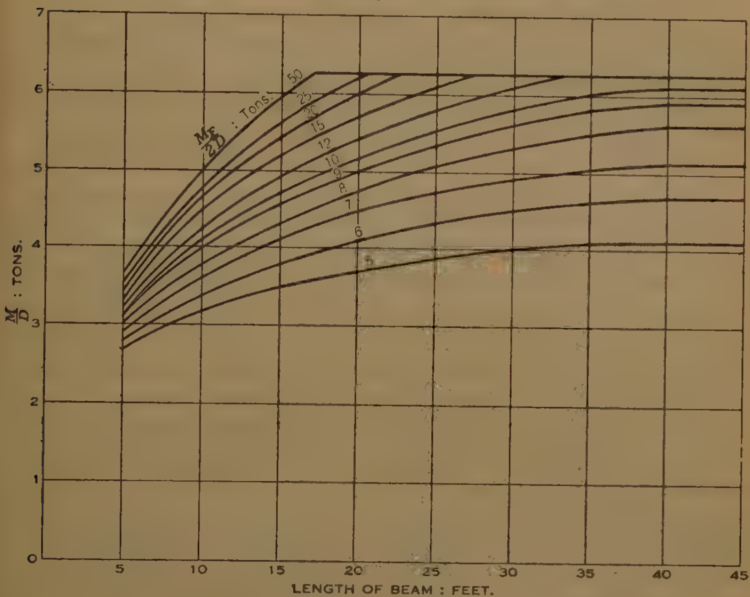
(and the same can in all probability be said of most welded connexions) are capable of transmitting large moments, with the result that appreciable restraining moments are developed at the ends of a loaded beam and that the corresponding moments developed in the stanchions are many times greater than those usually taken into account. This means that the stress-calculations made in design to-day give an inaccurate representation of the distribution of stress.

While this misrepresentation is more serious in the case of stanchions, it will be well at this point to deal shortly with beam-design, which is relatively straightforward. We have seen that in the actual buildings beam-to-stanchion connexions restrain the ends of the beams and so tend to reduce the maximum bending stresses in these members. The magnitude of this reduction for any applied load depends, of course, on the type of connexion and on the flexibility of the stanchion to which the connexion is made. When it was realized that the flexibility of the stanchion compared with that of the connexion had a relatively small influence on the maximum beam moment it was possible, by assuming a lower limit for the rigidity of the stanchion and by conducting a comprehensive series of tests on standard connexions, to draw up Tables and curves showing what minimum restraining moment could be depended upon in any beam fitted with a known connexion, no matter what the rigidity of the stanchion so long as it was above the lower limit. The curves and Table appropriate to a riveted flange-cleat connexion having a top cleat 6 inches by 4 inches by ½ inch are shown in *Fig. 7*. These data were prepared by Professor Cyril Batho, of Birmingham University, as a result of tests carried out by him. The restraining moment M can be found from them for any length of beam fitted with this connexion when D, the depth of the beam, and the loading as defined by M_F, the mean of the fixed-end or encastré moments, are known. With this information the design of the beam is as straightforward as that usual to-day. The maximum flexural stress in the member is easily calculated, being that which would occur were the ends freely hinged minus M, the restraining moment supplied by the connexion and found from the curves or Table.

It is when the condition of a stanchion is considered that the unsatisfactory nature of the present design-method is fully realized. Those of you who are familiar with the London County Council Code of Practice, or with the British Standard Specification No. 449 for the use of structural steel in buildings, know that the permissible stresses in stanchions tabulated there are deduced from the case of a pin-ended strut but that the designer, anxious to obtain some credit for the continuity of the stanchion through many floors, is allowed to consider the length of the member, not as the floor-to-floor height,

but as some smaller effective length. The only excuse for this reduction is the fact that the floor-beams framing into the stanchion provide some restraint. This postulates a connexion of some rigidity between the beam and the stanchion, and yet the bending moments,

Fig. 7.



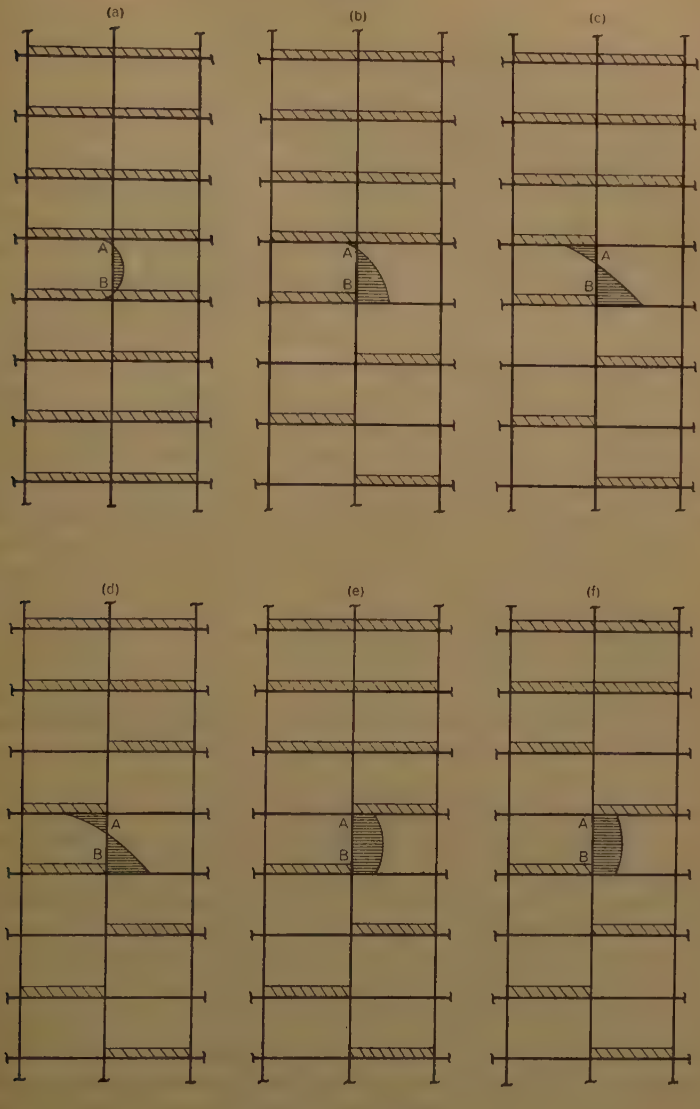
$\frac{M_F}{2D}$: tons.	Length of beam : feet.				
	5	10	20	30	40
6	2·8	3·4	4·1	4·5	4·7
8	3·0	3·8	4·8	5·3	5·6
10	3·1	4·1	5·1	5·8	6·1
15	3·3	4·5	5·7	6·25	6·25
20	3·4	4·7	6·0	6·25	6·25
50 —	3·6	5·0	6·25	6·25	6·25

coming into the ends of the stanchion-length from the beams, are calculated on the assumption of hinged joints. There is, in fact, a serious confusion of thought when dealing with compression-members.

It is not here alone that confusion is evident. As serious a matter is the fact that no real attempt is made to-day, even within the

imperfect limits of the usual assumptions, to assess the strength of the structure in relation to the purpose it has to fulfil. Every designer will claim that his object is to produce a structure of adequate strength and stability, that is, one to stand up to any loads which can legitimately be applied to it. To do this it is usual for him, in making strength-calculations, to consider all floors to be completely loaded to their full intensity. Are the conditions produced the most rigorous for all members ? This should be the first question the designer asks himself, and he might answer it by studying *Figs. 8*. It shows, in the left-hand top corner, *Figs. 8* (*a*), a symmetrical frame having all floors loaded. Consider the centre-length of stanchion AB. It will be subjected to an axial compressive force arising from the loads applied to the floors above. As the member is not perfect there will be a tendency for it to bend under this axial load, a tendency which will be resisted at the ends to some extent by the beams and connexions framing into the stanchion at A and B, so that the bending moment is of the form shown hatched in the diagram. This restraint, provided by the beams, and the resulting form of the bending-moment diagram, gave rise to the artificial and contradictory assumption of the existing Code of Practice relating to effective pillar-length, which I have just mentioned. However, returning to the bending-moment diagram, it will be seen that the maximum stress in AB is the sum of the axial compressive load per unit area and the maximum compressive stress due to the bending of the member. Now let us consider once again the actual bending stresses observed in the buildings tested, illustrations of which are given in *Figs. 4*, *5* and *6*. It will be remembered that when load was applied to the beam on one side of a stanchion appreciable bending stresses were developed in the stanchion-lengths both above and below. If, therefore, load is removed from the beam to the right of B the bending-moment diagram for AB will be of the form shown in *Figs. 8* (*b*), and the maximum bending stress will be greater than when all floors were loaded. The axial compressive load per unit area is, however, unchanged since no load has been removed from the floors above, so that the maximum stress in the stanchion-length is greater and the second arrangement of loading is more rigorous than the first. This arrangement may not produce the worst stress-conditions for the stanchion-length since if load is removed from the floor to the right of A a further increase of bending stress will occur, *Figs. 8* (*c*), and the decrease in axial end load may not compensate for this increase, leaving the maximum total stress still greater. It may be noted also that the bending stresses are further increased if load is removed from the next floor above on the other side of the stanchion, *Figs. 8* (*d*). In these last three arrangements of load the

Figs. 8.



bending moments applied by the beams to the ends of the stanchion-length bend it in double curvature.

Another and most important condition has to be considered in which the stanchion-length bends in single curvature. An arrangement of load which brings this about is shown in *Figs. 8* (*e*), load

being removed from the floors at A and B on opposite sides of the stanchion so that, due to the bowing of the member, the maximum stress will occur at, or near, the centre of length. As in the case of double-curvature bending the removal of a further load from the floors above, *Figs. 8* (*f*), will increase the bending stresses, while decreasing the axial stress, and it is possible that one or other of these arrangements which bring about single-curvature bending will produce the worst stress-conditions in the stanchion. It is impossible to say from inspection whether double- or single-curvature bending will produce the maximum stress, but it is certain that one or other will give a greater value than the symmetrical arrangement of load hitherto used. Every engineer will agree that it is much more reasonable to base the design on the worst condition possible than to consider some particular case, not the worst, and then to extrapolate, as it were, by the use of a large safety-factor in the hope of covering the worst. If the more reasonable way is taken the design of a stanchion resolves itself into two steps, (1) a determination of the end reactions applied to the member by those arrangements of load which produce the worst conditions in the member under both double- and single-curvature bending, and (2) an estimate of the maximum stresses developed by those reactions which will enable the suitability of the member to be judged.

Since under ordinary commercial conditions the design of a building-structure has to be produced in a comparatively short time it would not be helpful to leave the matter here. The first step in the production of a method of design must therefore be the collection of data which will enable the worst end-reactions to be estimated with ease. The magnitudes of these reactions are affected by the proportions of the members making up the frame and by the characteristics of the connexions joining the members. The tests on existing buildings, to which I have referred, showed that the introduction of floors and stanchion casing increased the rigidity of the connexions very considerably. Since the moments at the ends of a stanchion-length increase as the rigidity of the connexions increases, it was clear that for the normal type of clothed steel frame it would be necessary to assume, when dealing with stanchions, that the joints in the frame were perfectly rigid. Even with this assumption, however, any exact determination of the worst reactions could only be made if the proportions of all the members in the frame were known. As anything in the nature of a tentative design of the whole structure is out of the question, all the proportions cannot be known, so that the data to be provided must be such that an upper-limit value of the moment can be estimated from the meagre knowledge of the frame already possessed by the designer. As I have

indicated, the designer can determine his beam-sections before approaching the stanchions, but since the maximum moment developed in the stanchion, unlike that in the beam, is considerably influenced by the relation between the beam- and stanchion-stiffnesses, it is essential, if an economical estimate of the reactions is to be made, for the designer to choose a stanchion-section, as he does now in the existing method of design, and then to find whether his choice is satisfactory.

The calculations which had to be made before the data giving the most rigorous reactions could be set down were lengthy and no complete account of them can be given here, but some idea of the method used can be grasped from a consideration of one or two very simple cases.

Perhaps I should remind you that these end reactions arise from loading on the beams. The load is of two kinds: dead, due to the weight of the floor itself, and live, or superimposed, due to the weight of furniture and occupants. The first, that is the dead load, is found on every beam and cannot be altered, while the second can be arranged in many possible ways.

The experimental investigation had shown that accurate estimates of the moments in the structures could be made by means of the simple slope–deflexion method of analysis or by the Cross method, which is based on the same assumptions, so that it was possible to calculate the moments at the upper and lower ends of any stanchion-length. For instance, those in the fourth storey of a single-bay six-storey frame having a ratio of beam- to stanchion-stiffness of unity are given in Table II. They are arranged so that the moment

TABLE II.—MOMENTS IN INTERMEDIATE STANCHION-LENGTH: SINGLE-BAY FRAME.

	Beam 1 loaded.	Beam 2 loaded.	Beam 3 loaded.	Beam 4 loaded.	Beam 5 loaded.	Beam 6 loaded.
M_{34}	$= 0{\cdot}02M_F$	$+ 0{\cdot}08M_F$	$- 0{\cdot}39M_F$	$- 0{\cdot}13M_F$	$+ 0{\cdot}03M_F$	$- 0{\cdot}01M_F$
M_{43}	$+ 0{\cdot}01M_F$	$- 0{\cdot}03M_F$	$+ 0{\cdot}13M_F$	$+ 0{\cdot}39M_F$	$- 0{\cdot}08M_F$	$+ 0{\cdot}03M_F$

produced by the application of load to any beam can be picked out, and they are in terms of M_F, the fixed end or encastré moment, which has the value $\frac{wL^2}{12}$ for a uniformly-distributed load of intensity w per unit length on the beam. It will be seen from the Table that when the third floor is loaded the end moment M_{34} developed at the lower end of the stanchion-length is $-0{\cdot}39\,M_F$, while when the second is loaded it is $+0{\cdot}08M_F$. At the upper end the corresponding

moments M_{43} are $+0{\cdot}13\ M_F$ and $-0{\cdot}03\ M_F$. Our object is to find expressions for the worst possible moments that can occur in this stanchion-length. The worst moments are made up of two parts, one due to the dead load and the other to the most unfavourable combination of live loads. That due to dead load is found by adding algebraically all the moments, and has the value at the upper end (M_{43}) $+0{\cdot}45\ M_F^D$, where M_F^D is the fixed end moment due to dead load on a beam, while that due to live load is $+0{\cdot}56\ M_F^L$ found by adding the moments due to live load on beams 1, 3, 4 and 6, which are all of positive sign. As this is a single-bay frame, all stanchion-lengths must bend in double curvature, and these dead- and live-load moments give all the information required for this stanchion-length.

The nature of the bending to which the stanchion-length is subjected may be more easily appreciated from *Figs. 9* (facing p. 301), which show a six-storey two-bay frame, built of spring steel, carrying various arrangements of load. The frame is shown unloaded in *Figs. 9* (*a*). A and B are pointers rigidly attached to joints 4 and 3. The bending of the stanchion-length of the single-bay frame subjected to loads on all floors is illustrated in *Figs. 9* (*b*), where it can be seen, from the deflected form of the stanchion-length and even more clearly from the positions of the pointers which show that rotation of both joints has been clockwise, that bending is in double curvature.

In an internal stanchion the conditions are a little more complicated. This can be seen from Table III, which gives the end

TABLE III.—MOMENTS IN INTERMEDIATE LENGTH, INTERNAL STANCHION: TWO-BAY FRAME.

Bay No. 1.

	Beam 1 loaded.	Beam 2 loaded.	Beam 3 loaded.	Beam 4 loaded.	Beam 5 loaded.	Beam 6 loaded.
M_{34}	$+0{\cdot}02M_F$	$-0{\cdot}08M_F$	$+0{\cdot}39M_F$	$+0{\cdot}13M_F$	$-0{\cdot}03M_F$	$+0{\cdot}01M_F$
M_{43}	$-0{\cdot}01M_F$	$+0{\cdot}03M_F$	$-0{\cdot}13M_F$	$-0{\cdot}39M_F$	$+0{\cdot}08M_F$	$-0{\cdot}03M_F$

Bay No. 2.

	Beam 1 loaded.	Beam 2 loaded.	Beam 3 loaded.	Beam 4 loaded.	Beam 5 loaded.	Beam 6 loaded.
M_{34}	$-0{\cdot}02M_F$	$+0{\cdot}08M_F$	$-0{\cdot}39M_F$	$-0{\cdot}13M_F$	$+0{\cdot}03M_F$	$-0{\cdot}01M_F$
M_{43}	$+0{\cdot}01M_F$	$-0{\cdot}03M_F$	$+0{\cdot}13M_F$	$+0{\cdot}39M_B$	$-0{\cdot}08M_F$	$+0{\cdot}03M_F$

moments in the corresponding internal stanchion-length of a two-bay frame due to load on each beam of each bay. Here again the moments due to dead loading are found by adding all the moments. M_{43} has the value $-0{\cdot}45\ (M^D_{F_1} - M^D_{F_2})$ where $M^D_{F_1}$ and $M^D_{F_2}$ are the fixed-end moments due to dead load on the beams in the first and second bay respectively (their magnitudes are not necessarily the same). When the second and fifth beams of bay No. 1 are loaded, together with the first, third, fourth and sixth of bay No. 2, all giving positive values of M_{43} as can be seen from the Table, the end moments have their maximum live-load values ($M_{43} = 0{\cdot}11 M^L_{F_1} + 0{\cdot}56 M^L_{F_2}$) and the bending is in double curvature as can be seen from *Figs. 9* (*c*). If, now, beams 1, 3 and 5 of bay No. 1 are loaded together with 2, 4 and 6 of bay No. 2, the moment M_{43} has the value $(-0{\cdot}06 M^L_{F_1} + 0{\cdot}39 M^L_{F_2})$. This is smaller than the previous

TABLE IV.—TOTAL BENDING MOMENT DUE TO DEAD LOAD.

$\dfrac{K_{BR} + K_{BL}}{K_U + K_L}$	Bottom length : $\dfrac{\text{total moment}}{(M^D_{F_1} - M^D_{F_2})}$	Intermediate length : $\dfrac{\text{total moment}}{(M^D_{F_1} - M^D_{F_2})}$	Topmost length : $\dfrac{\text{total moment}}{(M^D_{F_1} - M^D_{F_2})}$
0·0	0·86	1·00	1·00
0·2	0·78	0·95	0·92
0·6	0·67	0·84	0·79
1·0	0·59	0·75	0·70
2·0	0·45	0·60	0·55
4·0	0·31	0·43	0·40
8·0	0·19	0·27	0·26

value but, as can be seen from *Figs. 9* (*d*), bending is in single curvature so that it may give rise to a higher stanchion-stress than the greater double-curvature moment. The two cases must be considered, each, of course, in conjunction with the dead-load moments. For all internal stanchions these three sets of moments, due to the dead load and to the arrangements of live load producing bending in double and in single curvature, must be evaluated.

It would be asking too much of the designer to calculate these moments in every case; in fact he would be unable to do so, as the full details of his frame are not yet known to him. Fortunately, it was found possible to collect safe values of the moments in any frame depending only on the limited data already possessed by the designer. This information, for dead load, is shown in Table IV. The first column contains the ratio of the sum of the stiffnesses K_{BR} and K_{BL}, of the beams to right and left of the stanchion, to

the sum of the stiffnesses K_U and K_L of the stanchion-lengths above and below the beams. The beams and upper stanchion-length are already designed, and a tentative choice will have been made of the lower length, so that the designer knows the value of this ratio. He can then read the total moment coming into the stanchion at the floor-level from the appropriate column in the Table according to whether a bottom, intermediate, or topmost stanchion-length is under consideration. The total moment being known, the moments in the upper and lower lengths are found by dividing the total between them in the ratio of their stiffnesses. The tests on existing buildings showed that this is not strictly justified, but it gives the best approximation possible. Table V gives the information for live-load moments; it is more complicated, since in intermediate and topmost lengths both double- and single-curvature moments must be included as we cannot yet judge which will be critical.

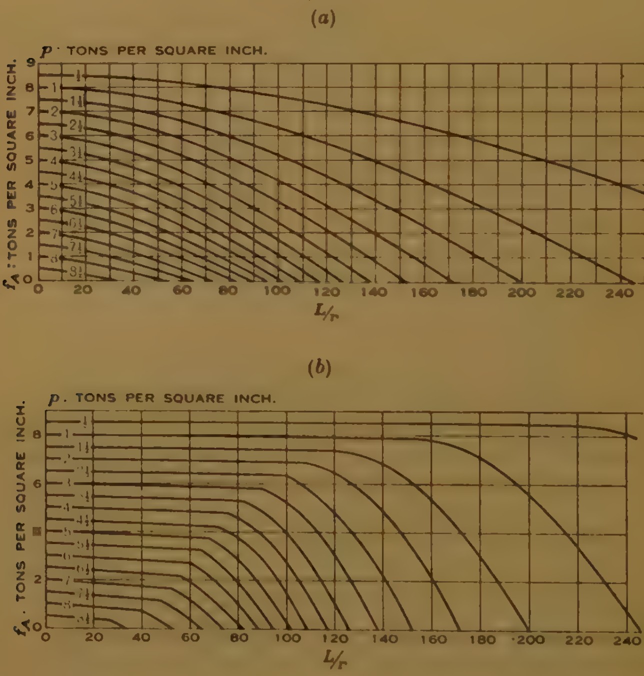
The reactions at the ends of the stanchion-length made up of axial load and end moments are now known, and the designer must test to see whether his choice of a stanchion-section is satisfactory. He must see that the maximum stress developed by these reactions does not exceed the permissible value. For this the families of curves similar to *Fig. 3* (p. 304) could be used, giving as they do the end bending stress which can safely be applied in conjunction with any axial load. You will remember, however, that the ratio of end bending stresses had also to be known, and to determine this the designer would have to consider tentative sections for two stanchion-lengths at a time. That is to say, in designing the length 43 a section for this length would be chosen so enabling the moment M_{43} to be calculated, but before the adequacy of this section could be tested M_{34} must be known and this means that a tentative choice of the length 32 would have to be made also. This might well be irksome, and it was therefore considered worth while to consider in detail only the moment at the upper end of the stanchion-length, which is under normal conditions the greater, and to assign a limiting value to the moment at the lower end such that safety is ensured, although inevitably at the cost of some economy. In this way only one ratio of end moments, and therefore only one curve from each family, is used. When the arrangement of load produces bending in single curvature a decrease in the bending moment at the lower end of the stanchion-length decreases the maximum stress in the stanchion. In this case safety will be ensured by assuming that the ratio f_B/f_A of the end bending stresses is unity, that is, by taking the moment at the lower end as equal to that at the upper. When the loads are such as to produce bending in double curvature a decrease in the magnitude of the moment at the lower end of the stanchion-

TABLE V.—TOTAL BENDING MOMENT DUE TO LIVE LOAD.

$\frac{K_{BR} + K_{BL}}{K_U + K_L}$	Bottom length.	Intermediate length.		Topmost length.	
	Double curvature.	Double curvature.	Single curvature.	Double curvature.	Single curvature.
0·0	$1{\cdot}15 M^L_{F_1} + 0{\cdot}29 M^L_{F_2}$	$1{\cdot}36 M^L_{F_1} + 0{\cdot}34 M^L_{F_2}$	$1{\cdot}00 M^L_{F_1} - 0{\cdot}00 M^L_{F_2}$	$1{\cdot}00 M^L_{F_1} + 0{\cdot}00 M^L_{F_2}$	$1{\cdot}00 M^L_{F_1} - 0{\cdot}00 M^L_{F_2}$
0·2	$1{\cdot}02 M^L_{F_1} + 0{\cdot}23 M^L_{F_2}$	$1{\cdot}23 M^L_{F_1} + 0{\cdot}28 M^L_{F_2}$	$0{\cdot}89 M^L_{F_1} - 0{\cdot}06 M^L_{F_2}$	$0{\cdot}92 M^L_{F_1} + 0{\cdot}01 M^L_{F_2}$	$0{\cdot}89 M^L_{F_1} - 0{\cdot}03 M^L_{F_2}$
0·6	$0{\cdot}83 M^L_{F_1} + 0{\cdot}16 M^L_{F_2}$	$1{\cdot}04 M^L_{F_1} + 0{\cdot}20 M^L_{F_2}$	$0{\cdot}73 M^L_{F_1} - 0{\cdot}11 M^L_{F_2}$	$0{\cdot}81 M^L_{F_1} + 0{\cdot}01 M^L_{F_2}$	$0{\cdot}73 M^L_{F_1} - 0{\cdot}06 M^L_{F_2}$
1·0	$0{\cdot}71 M^L_{F_1} + 0{\cdot}12 M^L_{F_2}$	$0{\cdot}90 M^L_{F_1} + 0{\cdot}15 M^L_{F_2}$	$0{\cdot}63 M^L_{F_1} - 0{\cdot}13 M^L_{F_2}$	$0{\cdot}72 M^L_{F_1} + 0{\cdot}02 M^L_{F_2}$	$0{\cdot}63 M^L_{F_1} - 0{\cdot}08 M^L_{F_2}$
2·0	$0{\cdot}52 M^L_{F_1} + 0{\cdot}06 M^L_{F_2}$	$0{\cdot}69 M^L_{F_1} + 0{\cdot}09 M^L_{F_2}$	$0{\cdot}47 M^L_{F_1} - 0{\cdot}13 M^L_{F_2}$	$0{\cdot}57 M^L_{F_1} + 0{\cdot}01 M^L_{F_2}$	$0{\cdot}46 M^L_{F_1} - 0{\cdot}09 M^L_{F_2}$
4·0	$0{\cdot}34 M^L_{F_1} + 0{\cdot}03 M^L_{F_2}$	$0{\cdot}47 M^L_{F_1} + 0{\cdot}04 M^L_{F_2}$	$0{\cdot}31 M^L_{F_1} - 0{\cdot}11 M^L_{F_2}$	$0{\cdot}41 M^L_{F_1} + 0{\cdot}01 M^L_{F_2}$	$0{\cdot}31 M^L_{F_1} - 0{\cdot}09 M^L_{F_2}$
8·0	$0{\cdot}20 M^L_{F_1} + 0{\cdot}01 M^L_{F_2}$	$0{\cdot}29 M^L_{F_1} + 0{\cdot}01 M^L_{F_2}$	$0{\cdot}19 M^L_{F_1} - 0{\cdot}08 M^L_{F_2}$	$0{\cdot}26 M^L_{F_1} + 0{\cdot}01 M^L_{F_2}$	$0{\cdot}19 M^L_{F_1} - 0{\cdot}07 M^L_{F_2}$

length may increase the maximum stress in the stanchion. For safety in this case the ratio f_B/f_A must be based on a consideration of the conditions which make the ratio of the magnitudes of the moments at the lower and upper ends of the stanchion-length the minimum. This minimum is found in the topmost length of an external stanchion when one topmost beam only is loaded and when the ratio of beam- to stanchion-stiffness $\left(\frac{K_B}{K_L}\right)$ approaches zero. The ratio of the moments at the lower and upper end then has the value

Figs. 10.



0·268, and this must be taken as the ratio f_B/f_A. The relevant information can now be embodied in only two sets of curves as shown in *Figs. 10*, the upper referring to single-curvature bending and the lower to double-curvature bending. They are, incidentally, based on a load-factor of 2; that is to say, they are so constructed that all the loads giving rise to the stresses could be doubled without raising the maximum stress in the member beyond the yield-stress of the material, taken to be 18 tons per square inch for mild steel.

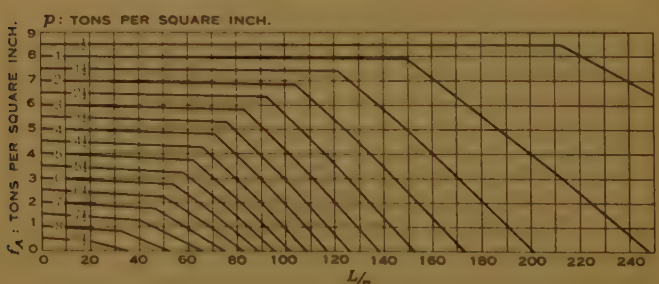
The design-procedure can now be summarized. Suppose an inter-

mediate stanchion-length is to be designed. A provisional section is chosen for it. The lengths above and all beams will have already been designed so that the ratio of the stiffnesses can be calculated. Concentrating attention first on the loaded beams at the upper end framing into the web of the stanchion, the end moments due to dead load and also to that live load producing double-curvature bending are found from the appropriate Tables, and the maximum flexural stress at the end of the stanchion-length produced by these moments combined is set down. To this is added the similar bending stress arising from the loading of beams framing into the flanges, to give the total maximum end bending stress. The next step is to see that the provisional stanchion-section can safely bear this stress together with the axial load per unit area imposed by the reactions at the ends of the beams above. The slenderness-ratio of the stanchion-length is calculated and the lower family of curves, *Figs. 10* (*b*), is brought into use. Each curve gives the relation between permissible end bending stress and slenderness-ratio for a particular value of axial load per unit area. If the axial load is, for example, 4 tons per square inch, attention is concentrated on the appropriate curve marked 4 and it is found that for the known slenderness-ratio, for instance 80, the permissible end bending stress, read from the vertical axis, is 4·6 tons per square inch. If this is greater than the actual total maximum end bending stress already calculated the section chosen for the stanchion will be adequate. It will be adequate to carry the dead load and the live load arranged so as to produce double-curvature bending; it may, however, be inadequate when the live load produces single-curvature bending. A similar calculation must be carried through, substituting for the double-curvature live-load moments those end moments which can be read from the appropriate Table (Table V) which are developed when the live load is so arranged that single-curvature bending is produced. When the resulting total maximum end bending stress has been calculated the appropriate curve from the upper family is used to test the adequacy of the stanchion-section.

Those of you familiar with the usual method of design will see that no fundamental change has been made, but the real magnitudes of the bending moments in the stanchion have been recognized, and by adopting rational strut-curves the consideration of unreal effective lengths has been avoided. There is one difference, however, which is of importance to the designer in his office. Owing to the necessity for considering both double- and single-curvature bending two independent tests of the adequacy of the stanchion-section have to be made in the new method. A number of engineers gave the method a trial in their offices and they were of the opinion

that as a result the time taken in proportioning a stanchion was prohibitive. Further simplification was therefore essential, so that, while the basis of the method was not undermined, one test only would be required for the stanchion section. This was made possible by producing one family of strut-curves made up of the critical portions of those already shown in *Figs. 10*. You will see from Table V that for a symmetrical frame the bending moment in double curvature is never less than 1·7 times the single-curvature moment. If, therefore, when considering single-curvature bending the magnitude of the live-load end bending moment is taken to be that of the double-curvature moment instead of the real single-curvature moment, and the test of adequacy is made on a family of curves similar to the upper family (that is, to the single curvature set (*Figs. 10*)) but having the ordinates increased 1·7 times, then safety will be ensured although some economy will be sacrificed. One calculation only of maximum end bending stress has in this way to be made, namely that for double-curvature bending, and if a composite family of curves is formed from the original double-curvature set and the new single-curvature set by taking that portion of each which gives the lower value of the permissible end bending stress, the work entailed in proportioning a stanchion is reduced by one-half. The composite curves are shown in *Fig. 11*,

Fig. 11.



and although I have only given you the argument for their adoption in the case of a symmetrical frame they are safe whatever the arrangement of the members.

This is a bare outline of the method of design from which you can appreciate its basis, but I have said nothing of what you will find on applying it.

The designer inevitably thinks of two questions: "Does it mean more work in the design office?" and "Does it result in any saving of material?" The first is easy to answer. Since there is little

doubt that the steps in the existing method were chosen mainly on account of their simplicity, it is inevitable that any more rational method must be more complicated. But let me suggest that you will not be in a position to judge the amount of work involved until you have had some experience of the method. The position is not as bad as the first perusal of the clauses of the Recommendations for Design [1] as published lead one to believe. The small amount of extra work is offset to some extent for the conscientious designer, who worries about these things, by the comfort in the thought that he is not asked to draw on his imagination to cover such points as the "effective lengths" of the stanchions.

The question of the weight of the structure is not so easily answered, bound up as it is with such factors as the intensity of superimposed load and the working stress. I have drawn your attention several times to the lack of truth in the picture of the stress-distribution given by the existing design-method. But let me remind you that the method as applied in Great Britain has resulted, as far as I know, in no building failure in more than 30 years. It must, therefore, include an adequate safety-margin supplied by a fictitious value for the superimposed load in the case of certain, though not all, occupancies, by the factor which makes the permissible stress a fraction of the yield-stress of the material and by others, such as those arising from the presence of clothing, which are still more or less obscure.

Although necessary, in part to cover ignorance when the existing method and Code are used, the factors should be modified for the new method, based as it is not only on a stress-distribution nearer the truth but on an arrangement of loads more trying to the structure.

Even so, it is clear that when using factors approximately the same as those embodied in the existing code the new method gives a saving on beams since account is taken of end restraint. The saving is considerable—for instance in carrying 1 ton per foot over a span of 16 feet a 12-inch by 5-inch by 32-lb. beam can be used instead of a 12-inch by 6-inch by 44-lb. beam.

Where stanchions are concerned, the emphasis in the new method is so different from that in the old that it is impossible to make a clear comparison. There will be a tendency for the distribution of weight in a continuous stanchion to change, the lower lengths becoming lighter and the upper lengths heavier. This is due to the recognition given to bending stresses, and this recognition means that where much the same factors are used the new method will call for heavier stanchion-lengths in some cases, though by no means in all. But the same factor should not be used. Not one of the

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[1] "Steel Structures Research. Recommendations for Design." H.M. Stationery Office, 1936.

innumerable stanchions designed by the haphazard method of to-day has caused any structural failure. If, therefore, the factor to be used with the new method is based on that actually existing in some of the more sorely tried columns now standing, then by adopting this more rational method throughout great economy must result, and framed structures will be free to evolve in a way impossible to-day.

CORRESPONDENCE
ON PAPERS PUBLISHED IN NOVEMBER 1936 JOURNAL.

Paper No. 5045.[1]

"The Restoration of the Breach in the Right Guide Bank of the Hardinge Bridge."

By Bertram Lionel Harvey, O.B.E., B.Sc., M. Inst. C.E., M.I.E. (Ind.).

Correspondence.

Sir Robert Gales observed that the Author suggested that the Sir Robert Gales.
ɔreach in the right guide bank which occurred on the 26th September, 1933, had had its origin at or just below water-level, and that it had ɔeen caused by a very strong suction on the face of the boulder ɔrotection, produced by "specially fast-moving eddies"; that uction caused silty sand, of which the bank was composed, to be lrawn out until the boulder protection fell down and exposed the ore of the bank to the action of the flood-water.

The slope of the bank was 1 in 2 and the sand was covered with a -inch layer of good stiff clay topped by 3 inches of quarry-refuse, he whole consolidated and held down by the weight of a 3-foot -inch covering of pitching-stone. Sir Robert had held an inquiry t the site in December, 1933, into the cause of the breach, and had ursued further inquiries in January, 1934, and although the need or some covering of the sand, to prevent its being sucked out or ven "running" out between the pitching-stones, had been realized rom the inception of "bund-and-apron" construction, no sug-uestion of that cause of failure had been made at the time of the ıquiries.

The breach had started between chainage 19 and chainage 22 of ıe guide bank. The original amount of stone pitching sufficient ɔr a scour-depth of 100 feet below low-water level had been supple-ıented by an addition of 400 cubic feet per foot run of guide bank, id in water at the toe of the apron, from chainage 15 to chainage 30. hat seemed to prove that the slip had not been caused by deepening ue to scour along the toe, and suspicion fell on the abrupt termina-on of the additional pitching at chainage 15. It seemed possible that

[1] Journal Inst. C.E., vol. 4 (1936–37), p. 21 (November, 1936).

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an eddy had been formed by that abrupt change of section and tha the resultant scour might have worked back to the point of origi of the breach. There was no evidence or suggestion of deepenir of the channel by scour along the toe, neither was there any evidenc of eddy-scour having occurred, and no definite conclusion wa reached.

A slip had occurred on the 11th January, 1934, at chainage 26·50 c the isolated head of the right guide bank, which showed as an exposu of the sand core of the bank above water-level. There was r question of scour at the toe or of eddy-action, as the river at tha time had been only 6 feet above low-water level and the curre had been slow. The slip was attributed to instability of the materi of the natural bank of the river, and it was suggested that th might have been the determining factor in the breach of the guic bank. No definite conclusion had, however, been reached as to tl cause of the breach. It remained for the Author, whose experienc of slips and their control was extensive, to suggest the true cause namely, that it was the result of local exposure of the sand core a about water-level, which he ascribed to fast-moving eddies.

Sir Robert thought that an explanation satisfying both of the case described above lay in wave-wash. In the case of the breach the was no need to go further than the Author's description of th waves that had rushed into the breach at intervals of about 2 minute accompanied by an afflux of about 2 feet in the embayment, and th small slip in the head of the guide bank was fully accounted for b the wash from the daily steamer-traffic which would pass that spo

Slips due to scour at the toe of a guide bank occurred with a fallin river, because the straight-running flood-water withdrawing in curving channels deepened them and the sand core of the guic banks was softened by seepage from the saturated banks. Th breach of the 26th September had occurred with a rapidly-risin river, but that no longer presented a difficulty. At the time, sandbank extended across the river from the left bank towards tl head of the right guide bank. As the river fell rapidly the curren slowing in the shallower water, had led to a rapid deposit of sand a silt and upward growth of the sandbank; but when the freshet h come down, accompanied by a still more rapid rise, it had found restricted channel opposite the guide-bank head, resulting in headi up and formation of the surface-waves which had persisted until tl channel had been re-opened to sufficient capacity. There was longer any doubt that those waves, by the suction they produc in passing along the face of the guide bank, had been able to dr out the covering of quarry-refuse and clay so as to allow the pitchi stone to drop and to expose the sand core, which had been swept aw

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by succeeding waves. As soon as the waves stopped, the embayment had ceased to progress towards the main line at the back of the bridge-abutment, and if the river had then begun to fall, the breach, which at that time had had no great depth where the slope-stone met the pitching stone of the apron, could have been repaired without much difficulty. The river, however, continued to rise, but it was not until the 2nd October, 6 days after the guide bank had been breached, that the water (which had been flowing into the embayment over the undisturbed stone apron at the back of the guide-bank head), by eroding the river bank at the edge of the apron, had begun to flow through the embayment and out by the breach in a volume which had threatened disaster. On the 5th October the river had commenced to fall, and on the 7th October it had fallen no less than 14 inches, bringing a rush of water at high velocities back into the main channel, of which the new channel at the back of the guide bank head became a part; a slight accident had thus led to a disaster. When the sand core of the guide bank had first been exposed, the damage could have been repaired in a few hours, and the breach as it existed up to the 2nd October could have been repaired in a few months, but, deepened and extended by the flowing through it of an arm of the main river, the breach had taken 2 years and had cost about £750,000 to repair.

Although there was no want of knowledge about the severity of the conditions to which the covering of the sand core was exposed at a guide bank in the lower Ganges, the design of a satisfactory covering had yet to be made. In the first bund-and-apron guide banks the reserve pitching stone was stacked on the slope of the bank, whence it went down and forward to supplement the apron stone where that proved to be insufficient. At the Curzon bridge over the Ganges at Allahabad it was formulated that the slope-stone should be treated as permanent, although it was considered that it should remain a flexible protection. The sand-core covering used there was 3 inches of quarry-refuse kept in place by loose stone pitching. The slope had never moved and the covering had proved satisfactory.

For the Hardinge bridge, assuming that the depth of scour for which the apron was to be designed was now known, it would be necessary to allow an additional strip of apron at the foot of the slope in order to ensure that there should be no movement of the slope-stone. If that could be depended upon, the safety of the sand core could be secured by a single layer of cut-and-fitted stone, as was found sufficient on the Suez canal to protect the banks from the heavy wave-wash of ships passing through it. However, even with the additional strip of apron at the foot of the slope, it seemed

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necessary to provide a flexible covering for the guide banks of th lower Ganges; he suggested that the covering should consist of 3-inch underlay of burnt brick on flat followed by 6 inches of broken stone ballast, and that the pitching stone should be hand-set and bedded down into the stone ballast to the extent that that could b done without the cost of cutting or breaking the pitching stone. The bricks were to provide a continuous cover for the sand, th stone ballast or other material was to protect the brick cover, th covering of stone was, by its weight, to keep the stone ballast in place, and it was to be hand-set to reduce the cavities through which the wave entered which produced the suction on withdrawal. Con centration of rainfall which would form gutters down the face was t be prevented by a clay banquette along the edge of the bank, which would ensure that all rain falling on the top of the bank would drain towards the inner slope.

The cost of the construction suggested above would have to b considered. The additional strip of apron to ensure as far as possibl the permanence of the slope would add to the cost of the guide bank but it would in fact form a much-needed margin of safety in the even of unexpected conditions showing the apron stone to be insufficien in quantity. The covering of good clay, often recommended, would be very expensive; the so-called clays in the delta were silt-deposit which, once dried, disintegrated in water. The famous Sara clay on the preservation of which the safety of the Hardinge bridg depended, was such a deposit, containing traces of wild rice; it had a small band of indurated material, but none of that would be avail able. A good clay was to be found in the Borind some 50 miles u the river, and a true clay, formed by decomposition of basalt, wa available at Phudkipur, 134 miles up the river where there wer stone quarries. Either of those clays would cost as much as quarry refuse. Quarry-refuse would be difficult to obtain in the quantitie required, and would probably have to be manufactured by crushing that led to broken stone ballast, and the size of the top 3 inches should be big enough to bridge the spaces between the pitching stones There would be no saving in pitching stone, as nothing less tha 3 feet 6 inches of pitching stone would be sufficient.

There were other considerations in connexion with the protectio of the face slopes of the guide banks of the lower Ganges. Owing t the vast amount of excavation which it would usually be necessar to do in about half a working season in order to lay the apron a low-water level, the floor-level of that part of the apron nearest th slope of the guide bank would in most cases be found to be at abou half-flood level, and it was only for the upper half of the slope tha it was possible to protect the sand with the covering describe

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above. Below half-flood level the only protection possible when the apron had gone down into place was the increased cover of pitching stone provided, which had hitherto been insufficient. The upper half of the slope was exposed to flood wave-wash and moving eddies, and therefore needed the greater protective covering, but the lower half was exposed to steamer-wash, which partially accounted for the slip described above in the head of the right guide bank. The safeguards that had been devised against that kind of slip were the reserve strip of apron already suggested, and a considerable increase in the thickness of the inner belt of the apron.

Mr. Inglis.

Mr. C. C. Inglis, of Poona, stated that model experiments had been in progress at Khadakvasla hydrodynamic research station, Poona, since the beginning of 1935 in connexion with safeguarding the Hardinge bridge. The main experiments had been planned

(*a*) to find out how scour occurred around a pier, the factors which affected the scour, and the best method for protecting the piers;

(*b*) to determine the angle of repose of various kinds of pitching in various conditions of flow;

(*c*) to observe how an apron (laid horizontally at low-water level) launched to form a pitched slope under various flow-conditions;

(*d*) to determine the exact conditions which led to the breach, and the best way to prevent a repetition.

A model of 25 miles of the river, to a horizontal scale of 1 to 500 had also been constructed, and was run with discharges of up to 20 cusecs, eleven series of experiments having been carried out to determine the effect of various control-designs as compared with the conditions in 1933. Subsequently the model had been enlarged to comprise a length of 60 miles of the river. He did not propose to describe those experiments, but merely to utilize the results to answer points raised in the Paper.

In *Fig. 2* (p. 26 §) the Author compared conditions in 1933 with average conditions over a period of years. That gave a wrong impression, because in most years there were rises and falls. On p. 31 § he stated that "it appears as if the main force of the river, swollen by the floods in Upper India, had concentrated above the bridge in the form of almost a tidal wave or waves with a steep hydraulic gradient, which lasted over a period of about 12 hours." Such a state of affairs was physically impossible, and an examination of the data did not indicate that there had been any phenomenal

§ Page numbers so marked refer to the original Paper. (Journal Inst. C.E., vol. 4 (1936–37) (November, 1936).)—Acting Sec. Inst. C.E.

Mr. Inglis. conditions, although the flood had undoubtedly been a heavy one and the quick drop had put a severe test on the stability of th pitching. It was found, however, from model experiments that th conditions were favourable to a slip and that the breach was to b expected even with normal discharges in the river.

On p. 27 § the Author stated that "investigations as to the caus of the breach have been exhaustive and, although various reasonabl and probable theories have been suggested, the actual and definit cause of what happened on the 26th September, 1933, ... will pro bably remain an unsolved problem." Every feature described in th Paper had been reproduced closely in the large model of the river and a model designed to determine the exact way in which th breach occurred had reproduced the breach accurately. It had, i fact, been found possible to cause a breach at different parts of th guide bank by a suitable adjustment of the conditions.

When Mr. Inglis had first visited the bridge, there seemed to be general opinion that the breach had been due to the large-radius slow-moving eddy, which revolved slowly in the vicinity of th breach, the idea being that the velocity of the eddy increased wit depth, causing a hole to form near the toe of the pitching, and th Author referred to "strong eddy suction." Neither of those explana tions was correct or even physically possible; the true explanatio —as viewed through a glass sheet let into the side-slope of a mode of the right guide bank—was that below the slowly-revolving surfac eddy there was a region of instability at mid-depth, where the flow was sometimes in one direction and sometimes in another; but th only severe action was at the bed, at the toe of the pitching, and ther the flow was straight and turbulent, coming in surges, some stronge than others, at the rate of from one to three per second in the model Sometimes sand was picked up in little gusts like dust on a road Stones were carried away in the surges, and although a stone migh remain unmoved for several seconds, it might then be carried awa in a powerful surge. Experiments with a model of a pier had show that silt was scooped out around the pier by diving curved flow and that stones thrown into the "cups" so formed usually remaine there, gradually sinking into the bed as scour progressed. Hence although no stone might be visible around the pier, it was merel because it was covered by a layer of silt. Stone thrown or lai outside the "cup" was washed downstream and served no usefu purpose; it might even form a local obstruction and be a source o danger.

The Author on p. 25 § gave the reasons for the construction of th

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§ *Ibid.*

Damukdia guide bank. Mr. Inglis wished to make it clear that the action taken was what "common sense" would indicate to be the safest course, and hence it was easy to understand why that operation was followed in an emergency. The action taken, however, ignored the first principles of river-control—namely, that opposing the direct force of flow caused relatively harmless kinetic energy to be changed into highly-destructive turbulent eddy-flow of an unstable kind, accompanied by surging, and that the only way to control flow was by "coaxing" the river to swing in a large, natural curve, around a fixed guide bank. Constructing the Damukdia guide bank ignored the first principle directly, and also prevented the right guide bank functioning in the way it was designed—namely, as a Bell bund. Had the right guide bank been constructed of the correct length in the first instance, it seemed fairly certain that the local engineers would have let well alone, but owing to its being much too short their anxiety was easily understood. So long as the river flowed from Sara to form a loop around the right guide bank, the bank was of ample length; it would only be of insufficient length if attack came from the right, upstream, instead of from Sara on the left, as at present, and there was no possibility of attack from the right bank. That followed from the reasoning of Bell and others; but the exact effect had been examined in the model, which showed that had the Damukdia guide bank not been constructed, the loop of the embayment would not have extended below the end of the guide bank, nor eaten more than 2,400 feet into the right bank. Some conclusions, drawn from the experiments, which had a direct bearing on questions raised in the Paper were: Mr. Inglis.

(i) A guide bank tended to pull a river around its nose, but there was a limit to the embayment caused thereby.

(ii) Flow tended to "hug" guide banks and to leave them with reluctance.

(iii) Scour of 206 feet had been recorded near Sara. That was considereably deeper than pier-foundations. Pitching around piers retarded action but did not prevent it. The proper place to protect piers was, therefore, upstream—by the correct design of the guide banks.

(iv) After a "falling apron" had launched, so as to form a pitched slope, action would begin only at the toe, not on the slope, and when the whole of the stone of the falling apron had been launched, further scour at the toe would lead to stone being denuded from the original slope above water-level, leaving part of the bank exposed below water-level. The area of slip would increase rapidly unless localized by the method described by the Author—namely, ". . . to pitch boulders speedily over a length of about 15 feet beyond

Mr. Inglis. the slip, at both ends, at water level," while simultaneously covering the exposed area of the slip by throwing boulders from above.

(v) The retired guide bank at Sara had caused an abrupt change in direction of flow, which induced diving flow, leading to a scour 206 feet deep at high flood. That had had the effect of directing unstable surging flow towards Damukdia on the opposite bank. When the Damukdia guide bank was constructed to oppose the latter flow, it prevented free embayment by natural scour, which would have dissipated a large amount of energy and would have smoothed out instability of flow. It had also increased instability and surging, and so had intensified the attack which caused the breach.

(vi) The basic principles of Bell and Spring were sound, but they required revision and amplification in the light of recent knowledge. Work of that kind could not, however, be reduced to rule-of-thumb methods, and every case required special consideration. Bearing in mind that a mistake in design might necessitate the expenditure of hundreds of thousands of pounds, it was obvious that designs should be prepared by construction-engineers and river-training specialists working in collaboration. Some years ago, model-experiments were looked upon with suspicion, but undue reliance was now frequently placed on them. Both attitudes were unsound ; a river-model did not yield geometrically-similar results to the prototype, and so the design of models and the interpretation of results presented problems of great complexity and difficulty. It was, in fact, necessay to construct different models to determine the effect of different factors, and then to combine the various results in a large river-model (with a minimum discharge of 7 cusecs), from which accurate results could be obtained.

Mr. Leitch. Mr. W. O. Leitch observed that a liberal amount of rubble had been laid in the moles, but a large proportion of it appeared to be "one-man-rock" which, should movement occur, seemed hardly heavy enough to prevent the stones from being rolled away by the current, instead of then sinking down and possibly filling the first small scour-hole.

The permanent protection of the bridge was not the main subject of the Paper, but as a considerable time had elapsed, perhaps the Author could say what was about to be done to deal with such deep scours as 120 feet, which, having occurred once, might occur again, and which would seem to make it necessary to tackle the very expensive problem of training the river for a greater distance above the bridge, so as to steady the current down to and past the bridge.

Mr. Radice. Mr. W. A. Radice, of Calcutta, observed that important decisions affecting the safety of the bridge had had to be made quickly, and

he wished to express his admiration of the soundness of judgement displayed by those responsible. Mr. Radice.

When he had visited the restoration works in 1934 the mole was practically completed and the stone dykes of the backwater-bund were just appearing above the water. He had then expressed to the Author the fear that, as the river rose in flood, and perhaps even more so as it subsided, the changes in river-levels might cause the water in the lagoon between the mole and the backwater bund to act as a lung, and to breathe out pulsations which, impinging on the main current flowing past the mouth of the lagoon, might cause swirls dangerous to the safety of the outer downstream face of the mole. He would be interested to know whether any such tendency had developed during the rise and fall of the river in 1934, because if no inconvenience or danger were caused by the presence of the lagoon, the fact might have an influence on the existing practice of design of guide banks, and might lead to a revision of theories on river-training and the need for unbroken continuity of the protective works.

During that visit he had also noticed that the space between the back of the mole and the earthworks of the right-bank approach to the bridge itself formed a deep enclosed hollow. The Author had informed him at the time that the question of providing means to flood that hollow to correspond with the river-level outside, so as to equalize the pressures on each face of the mole, was under consideration. It would be of interest to know what the decision had been, and its effect on the behaviour of the mole during the ensuing flood, especially with regard to seepage through the mole itself and the disposal of rain-water accumulations in the hollow.

The Author, in reply, observed that he was very grateful to Sir Robert Gales for the very thorough and detailed remarks he had made, and for agreeing that the original slip might have been caused by the local exposure of the sand core at about water-level. The Author continued in charge of the Hardinge bridge for the remainder of the flood season, following the completion of the Paper, and the slips that occurred during that period had strengthened his conviction that slips, if left uncared for, would invariably turn into breaches. Mr. Inglis, on the other hand, stated that from the observations on his model-experiments he had found the cause of the breach to be due to other reasons—in fact, by a suitable adjustment of conditions in the model it had been found possible to produce breaches at different parts of the guide bank. The Author had had the good fortune to witness the earlier series of experiments which Mr. Inglis had carried out in his laboratory, but during those experiments it had seemed to the Author that a breach, by any The Author.

The Author. other method than undermining the toe, was impossible as the guide banks were constructed of a solid clay material, very nearly of the consistency of concrete. He was, however, glad to learn that apparently subsequent to his visits, the model had been altered to reproduce closely every feature of the guide bank in the prototype and that Mr. Inglis had been able to come to the conclusion he had stated. Nevertheless, the Author maintained that a slip, if left unattended, would develop into very much larger proportions and would grow until it became a breach; it was also a possibility that a breach could occur by undermining at the toe of a fully-fallen apron. The additional strip of apron at the foot of the slope suggested by Sir Robert Gales would be a good feeder for slips or breaches which commenced lower down, but if slips commenced, as so very many did, above low-water level, or, worse still, at about high-flood level, the extra strip of apron would not function in the way that was intended.

With regard to the Damukdia bund, it was unfortunate that at the time it had been designed and constructed the railway had not had the advantage of the knowledge which Mr. Inglis had obtained from his model, as then a large amount of money and anxiety would have been saved. It was, however, built after the considered opinions of very many engineers had been taken and was now, the Author understood, being extended.

The model experiments carried out by Mr. Inglis had, in the Author's opinion, given, and would continue to give, most valuable information, with special reference to specific details, such as the pitching of piers, the launching of aprons, the maximum scour in a certain channel, and other such localized subjects which were badly in need of investigation. The Author, however, felt that, over an extended length of 50 miles of an alluvial river, where so many factors entered into the picture and where the alteration of even one factor would perhaps cause innumerable changes downstream, the investigation of the behaviour of a river like the Ganges, by means of a model, was not perhaps the best method; he felt that experience, together with frequent inspection and observation, combined with common sense and an application of general hydraulic principles, could supply the knowledge necessary for designing and maintaining training works of large rivers, aided perhaps, in matters of detail, by the knowledge acquired from the results of model experiments.

Mr. Leitch's remarks were interesting because the Author understood that work was now being carried out to protect the river-bed from scouring to even greater depths than it already did in the vicinity of the guide banks, and thereby to avoid those banks being exposed to the action shown through the glass side on Mr. Inglis'

model. That was being done by continuing the fallen apron far out into the stream in the form of a carpeting or apron from 3 to 4 feet thick in extension of the fallen apron. The work was both slow and difficult, but it was successful if it were done carefully through floating grids, and by the method described on p. 38 §. "One-man-rock" had been confirmed by the model-experiments to be a satisfactory size of pitching stone. If the rocks were cubical and irregular they were not very easily detached from a mass and rolled away, except in exceptional circumstances, or where the silt below them had been scoured. **The Author.**

Mr. Radice's questions and remarks were particularly interesting because they referred to phenomena that Mr. Radice had discussed as probabilities at the time that the work was in hand, and because the troubles which Mr. Radice had anticipated had actually occurred. In the case of the impounded water behind the mole the trouble had come very early in the flood, in July in fact. The pumping arrangements for the equalization of water-level between the inside of the mole and the river had not been completed by that date, and the seepage he referred to had occurred, causing great damage to the earth bank of the mole and necessitating emergency pitching on its slope with bags of earth and the surplus stock of undersized boulders. Realizing, after that had occurred, that it might occur again at higher levels later, the same protection had been carried up to above high-flood level. When the pumping arrangements had been completed, and were in operation night and day whenever necessary, a maximum difference of 6 inches in the relative levels had been maintained through the flood and late into the fall of the river.

The other trouble that Mr. Radice had feared had also occurred, with very nearly disastrous results. The main stream, flowing past the sheltered water in the lagoon, had set up a very unpleasant large slow-moving eddy. Slips had occurred, probably due to that eddy, both at the river end of the back-water bund and just around the corner, on the river side, from the end of the mole. It seemed, therefore, that the unbroken continuity of guide banks, at bridge abutments, would have to remain the correct design for protective works.

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§ *Ibid.*

Paper No. 5034.[1]

"Newry Ship-Canal Improvement Scheme."

By Robert Ferguson, B.A., B.E., M. Inst. C.E.

Correspondence.

Mr. Marsh. Mr. C. M. Marsh observed that the conditions on the Newry canal had many parallels in the Weaver Navigation in Cheshire. Both systems were combinations of docks and a ship-canal, but while at Newry the docks were at the upper end of the ship-canal, on the Weaver the docks were situated at Weston Point on the Mersey and the canal acted as a feeder running inland to the chemical works at Northwich (13 miles) and to the salt-works at Winsford (20 miles), and was available for coasters with fixed masts and funnels for a distance of 16 miles.

The figures given on p. 55 § as the maximum dimensions of boats likely to use the Newry canal were almost identical with the limits at Weston Point docks (namely, 240 feet length by 15½ feet draught by 37 feet beam); as the Author was providing for vessels of 1,000 tons gross with those dimensions, it might be of interest to record that at Weston Point the largest steamer docked during the last 10 years had a gross tonnage of 1,436, and that during that period as many as fifty-two vessels of over 1,000 tons gross, carrying on an average 1,400 tons, had been docked.

The narrowest places on the old canal at Newry were stated to have had a waterway area of 600 square feet. That was a similar figure to that for the narrowest places on the Weaver Navigation, and gave a waterway-to-craft ratio of 3 to 1 for typical loaded Weaver craft. That ratio was found to produce serious bank-erosion, and wherever possible steps were being taken to increase the area of the waterway. The selection of 3 to 1 on the Newry canal was therefore of interest, as was the adopted profile with underwater slopes of 1 in 1½, which would appear to be unusually steep unless the strata through which the Newry canal passed were exceptionally favourable. Examination of recent cross-sections on the Weaver showed normal slopes under-water of 1 in 2½, whilst a slope as steep as 1 in 2 was only occasionally found. The typical cross-section was saucer-shaped with slopes steepest at mid-depth and flattening out towards the surface and towards the bed of the canal.

No mention appeared to be made in the Paper of the speed of

[1] Journal Inst. C.E., vol. 4 (1936–37), p. 51 (November, 1936).

§ Page numbers so marked refer to the Paper.—Acting Sec. Inst. C.E.

vessels navigating the Newry canal, or whether it was anticipated that vessels of large dimensions would pass each other in the canal. The bed-width of about 50 feet would indicate that passing was not intended, at any rate with steamers of 28 feet beam; in view of the relatively short length of the canal it would be a simple matter to avoid that without serious delay to vessels. If that were so, a more stable profile would seem to have been possible by easing the side slopes, with a reduction in the bed-width, the surface width being apparently the limiting factor. Mr. Marsh.

The Author's remarks as to the longevity of timber that was completely immersed could be confirmed by numerous examples on the Weaver. During the recent construction of a flood-water sluice, a disused lock, constructed in the 18th century, was pumped out. The floor of the lock was formed of timber baulks, rough dressed, about 12 inches square, spaced 4 feet apart with the intervening spaces filled with hand-packed rubble. Those timbers were without exception in perfect condition. The standard method of bank-protection, which had been used on the Weaver for 50 years wherever there was shallow water alongside the bank, was by overlapping pitch-pine planks. The present condition of the oldest planking was therefore of interest: the upper parts above water level were badly decayed, whilst the portions entirely immersed and those in the ground were in perfect condition. The present method of treating that planking was to drive the timber sheeting so that the tops were only just above water-level: they were provided with a mild-steel flat waling anchored to back piles at 7-foot 6-inch centres and were entirely encased in a concrete capping extending about 3 inches below water-level. The spacing of the back piles and tie-rods was worth noting in view of the Author's spacing of 30 feet between centres. A spacing of 15 feet between centres had been originally selected on the Weaver, but bulging of the planking had occurred; intermediate rods and piles were added and for about the last 40 years a spacing of 7 feet 6 inches had been used with complete satisfaction. The cost of the present form, complete with concrete cap, tie-rods and back piles, together with a moderate amount of filling and bank sloping, averaged 45*s*. per linear yard, which compared with the 38*s*. 6*d*. per linear yard given in the Paper.

The details of the construction of the outer gates at Victoria lock were of considerable interest and value. The most radical change made was the replacement by steel buoyancy-gates of the timber gates which had had a life of 80 years. It would be of interest to know whether the provision of timber gates—greenheart frames with pitch-pine facing—had been considered, and to have the reasons for the selection of steel gates. Did the Author anticipate that

Mr. Marsh. the steel gates would have a life of 80 years? Mr. Marsh regretted the absence of information as to the steps taken for obtaining accurate templates for dressing the heelposts and clapping-sills of the new gates to suit the old masonry, and as to the difficulties experienced in fitting the new gates. In view of the time during which the canal was out of commission, it would seem probable that it had been found necessary to remove the old gates prior to the final dressing of the timbers of the new gates. Could the Author confirm that point, and had any information been gathered from the course of the work which would have enabled the time to be reduced?

The Author. The AUTHOR, in reply, noted with interest the similarity between the dimensions of the entrance lock at the Weston Point docks of the Weaver Navigation and those proposed at Newry for the accommodation of vessels of 1,000 tons gross. The fact that a number of vessels up to 1,400 tons gross had docked at Weston Point did not preclude, however, the possibility of one of the dimensions of a vessel of 1,000 tons gross from reaching the limits of any one of the three dimensions of the lock. In order that a vessel of 1,000 tons gross might be chartered without special reference to its length, beam, or draft, it was necessary to provide for the contingency of such a vessel being somewhat abnormal in one of its dimensions.

The material through which the canal had been cut, although varying in character was generally favourable, and in the vicinity of McShane's bridge, where the greatest widening had been effected, it consisted of hard gravel which was extremely difficult to excavate. Originally, the banks had been pitched to a depth of about 3 feet below the water-level of the canal and the underwater slopes below that depth had been unprotected. The top portion of the bank had been formed of imported material which, even when pitched, did not stand up to the wave-action and required constant repairs, but the lower underwater portion, although standing unprotected at a slope of 1 in 1½, had been stable for almost a century. Artificial waterways with unprotected slopes, although usually constructed to a trapezoidal cross-section, generally reverted to the saucer-shaped section of a natural channel, due no doubt to the elbow between the bed and the slope being filled with material eroded from the bank.

The distance between Albert basin and Victoria lock was about 3½ miles and vessels were required to take at least 1 hour in travelling that distance. A lie-by was provided in Ogle's Bay (*Fig. 1*, p. 52 §) to enable a vessel catching the tide to pass an incoming vessel, but normally the sailing-arrangements made by the harbour-master avoided that necessity. The bed-width had been made as wide as possible, consistent with the stability of the underwater slopes, with

§ *Ibid.*

The Author.

the object of increasing the sectional area of the waterway and not for the purpose of enabling vessels to pass.

The method of bank-protection adopted at Newry differed from that on the Weaver, as king-piles, the rubble septum, and the masonry wash-wall had not been provided on the Weaver. Accordingly, the spacing of the tie-bars on the Weaver ought to be much closer than was necessary at Newry.

The replacement by steel buoyancy-gates of the timber outer gates at Victoria lock was more economical in initial cost than the replacement in greenheart, necessitated the canal being closed for a shorter period of time, and obviated the underwater renewal of the existing cast-iron roller-path which was very much worn. Moreover, it followed the precedent that had been established in 1923, when the inner gates were replaced with steel buoyancy-gates. The Author did not possess the prophetic powers that were necessary to anticipate with any degree of accuracy the life of steel lock-gates, but he considered that the comparative ease with which steel buoyancy-gates could be floated out for major repairs and re-stepped conferred an advantage on that type of gate over the ponderous timber gates which could not be handled without the use of heavy lifting-tackle. In a small port like Newry that type of plant was not available and it was necessary in the dismantling of the old gates to cut the ribs into sections for easy removal. The operation of dismantling the old gates had occupied a considerable portion of the time that the canal was out of commission, and further delay had been caused by the rapid accumulation of mud which seriously hampered the diver in getting into proper position the template for the clapping-sills and socket casting. There had been no difficulty in dressing the heelposts to the radius of the hollow quoins, but the clapping-sills could not be dressed until the templates of the masonry sill had been obtained, and that was not possible until the old gates had been entirely removed. If there had been no mud-problem and if heavier lifting tackle had been available at the site the time taken in installing the new gates would have been reduced appreciably. No difficulty had been experienced in the actual stepping of the gates.

The operation of taking a template of the masonry sills presented no special feature. The diver screeded the face of the sill and projected the line on to the underside of the wood template, which was then cut out so that the edge of the template could be scribed to the face of the sill. As the cast-iron nosing had corroded, and in some places the masonry had been grooved by the action of the hauling chains, it was not possible to obtain a perfectly water-tight fit.

Paper 5062.[1]

"Simple Experimental Solutions of Certain Structural Design Problems."

By Professor Alfred John Sutton Pippard, M.B.E., D.Sc., M. Inst. C.E., and Stanley Robert Sparkes, M.Sc.

Correspondence.

Dr. Nichols. Dr. H. J. Nichols, of Bombay, considered that the method dealt with in the Paper was one which might be of some value in dealing with a small structure which relied for stability on the stiffness of its members against bending. As the size of the structure increased, the percentage error became of greater importance, and the necessity for accuracy became greater. In the structure dealt with in the Paper, however, the calculations themselves were not excessively long, and could be relied on to predict with fair accuracy—certainly within the limits of the tolerances of the British Standard Specification for Structural Steel—the stresses to be expected.

It could not be overlooked, however, that even in flat two-hinged arches the arch-shortening effect could be considerable and should be taken into account, and that that effect depended on the cross-sectional area of the rib. When the moment of inertia of a typical built-up steel member was represented in a model by a member rectangular in section, as was inevitable in models cut from sheet material, the sectional area of the model-member was always greater than its scale value. The low experimental value given on p. 92 § for the roof-portal for the new Central Depot (Westminster) exemplified the effects of that difference in area. On the other hand, in the case of the Lengue arch, in which the model-members were also rectangular in section and proportional to the sectional area of the full-size members, those members were bound to have been considerably deficient as regards their scale lateral stiffness.

The only real solution was to recognize the fact that each structural member was possessed of two separate fundamental properties, its lateral stiffness against bending and its stiffness against longitudinal strain, and to provide accordingly for those properties in models. That adjustment could be made comparatively simply by introducing into each model-member, proportioned for lateral stiffness, a device to reduce its longitudinal stiffness to the correct scale value. That device could be similar in principle to (although very much smaller and stiffer than) that described in Dr. Nichols's own Paper on "Pre-

[1] Journal Inst. C.E., vol. 4 (1936–37), p. 79 (November, 1936).

§ Page numbers so marked refer to the Paper.—Acting Sec. Inst. C.E.

tressing Bridge Girders."[1] That device, it might be mentioned, need not affect the transverse stiffness of the member in question. **Dr. Nichols.**

The method of model-investigation was one which had unfortunately been considerably neglected in Great Britain in the past, and its developments held out prospects of a more ready acceptance among British engineers of new forms of construction.

The AUTHORS, in reply, agreed substantially with the comments made by Dr. Nichols, but observed that his criticism applied equally to most other suggested methods of experimental stress-analysis. Unless the elastic properties of the structure were reproduced in exact proportion errors were bound to occur and their importance would vary with different types of structure. In many cases simplified models of the kind used by the Authors would introduce errors so small as to be negligible, and probably less than those caused by assumptions made in arithmetical stress-analysis. In the calculation for the roof-portal for the Central Depot, for example, the structure had been divided into a number of short lengths each of which was assumed to be of constant cross-section; the difference of 3·7 per cent. quoted in the Paper was therefore the difference between two approximate solutions. An exact solution would be so laborious as to be impracticable. **The Authors.**

The value of the Authors' method depended upon its simplicity and the ease with which results sufficiently accurate for many purposes could be obtained. A distinction had to be made between an elaborate experimental investigation such as that described by Dr. Nichols in his Paper[1] and a simple preliminary estimate or check on calculation, which was the object of the Authors of the present Paper. In many types of structure the ingenious device due to Dr. Nichols would be advisable or even essential, but to introduce it in every case would make experimental analysis sufficiently formidable to deter most designers.

Paper No. 5028.[2]

"Repeated Stresses on Structural Elements."

By Professor FREDERICK CHARLES LEA, O.B.E., D.Sc. (Eng.), M. Inst. C.E.

Correspondence.

Mr. H. B. GATES, of Perth, Western Australia, referring to the figures given in Table V (p. 109 §) in which it appeared that the safe **Mr. Gates.**

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[1] Journal Inst. C.E., vol. 5 (1936–37), p. 91 (February, 1937).

[2] Journal Inst. C.E., vol. 4 (1936–37), p. 93 (November, 1936).

§ Page numbers so marked refer to the Paper.—ACTING SEC. INST. C.E.

Mr. Gates. range of stress in joint type "O" was only about 8 to 9 tons per square inch at the limit, wished to point out that the bearing stresses used were too high. For a working tensile stress of 8 tons the bearing stress usually recommended was not more than 10 tons, whereas that employed in the specimen was 13·33 tons. Undoubtedly a high bearing stress complicated the stresses around the rivet-hole, thus giving a false idea of the tensile strength of the plate. He had always suspected that 10 tons was too high, and that the bearing stress ought not to exceed 8 tons.

Again, the transference of stress through a riveted joint was a complicated matter, and did not receive sufficient study from the average designer. The effect of an added cover-plate was to produce a discontinuity of section, similar to a notch. There was a sudden transference of stress at the first and last rows of rivets, whilst an intermediate row, if used, took only a small proportion. Those considerations led him to believe that the diamond form of cover was unsound, as there was a definite concentration of stress carried by a single rivet at the point. That conclusion was borne out by the curves in *Fig. 13* (p. 111 §) where joint "P" was seen to be by far the best. Similar reasoning applied to welded cover-plates. The general conclusion was that there should be no stinting in the number of rivets, or, alternatively, in the weld-metal.

Dr. Nichols. Dr. H. J. Nichols, of Bombay, wished to express his admiration for the way in which the Author had carried out the series of tests described in the Paper, but he was doubtful as to the extent to which the manner of loading the test-specimens represented actual loading conditions in a member of, for example, a bridge structure.

It was stated on p. 95 § that the repeated-stress experiments had been carried out in a Haigh machine, but information was not given as to the cyclic speed. Presumably the tests had been run at a comparatively high rate of repetition, and had proceeded continuously until failure had occurred. Such loading conditions were however, very different from those obtaining in any practical structure. A typical loading in, for example, a member of a railway-bridge truss might be represented by twenty repetitions of stress rising to a maximum and dying away again, followed by, say, one hour of complete rest for recovery under a moderate mean load, after which the twenty repetitions would be repeated, and so on. In Mr. Nichols's opinion a test carried out on the above lines would show vastly different results from those obtained by a continuous vibration. To carry out such a test would require a great deal of time and a thoroughly realistic test would evidently have to last as long as the life of a steel bridge, or perhaps 60 years ; nevertheless, useful

§ *Ibid.*

omparative data might be obtained in a reasonably short time by epeating tests which ended after about 4,000 repetitions when run ontinuously, and substituting intermittent loading as outlined bove. Dr. Nichols.

It was obvious also that all loadings of a bridge in service were not apacity loadings. A maximum load might occur once per month r once per year, whilst average working loads were little more than 0 per cent. of capacity. It would be of interest if the Author ould give his opinion as to what effect, if any, occasional high tresses were likely to have on the ultimate endurance of a member ubject to repetitions of stress just below the fatigue-limit. Could he Author also state whether, in the case of continuous repetition ests, there was any connexion between the percentage elongation f a specimen and the number of repetitions it would stand before ailure, and if so, could he suggest a practical formula to reduce safe vorking stresses in terms of the reduced percentage elongation ?

The Author, in reply, agreed that it was quite true that in repeated- tress testing, the conditions could not be the same as under the ctual conditions of working, but the same could be said of any cheme of testing with which he was familiar. The questions raised y Dr. Nichols, however, as to the cyclical speed and the effect of eriods of rest between the applications of cycles were of great mportance. Wöhler's original experiments had been carried out t slow speeds as compared with much recent work, but for wrought ron and mild steels it seemed to be well established that within a ery wide range the periodicity of the cycles did not affect the range ery considerably at ordinary temperatures. At very high fre- uencies, as shown by Jenkin at 30,000 cycles per second, the range vas higher than at, say, 30 cycles per second, but between 3 and 80 ycles per second it had not been demonstrated that there was any ppreciable difference in the fatigue-range. In the Haigh machine sed in the tests, the number of cycles per minute was 2,000. The uthor had carried out tests from 320 to 5,000 cycles per minute, ut he had not been able to establish a marked difference in the atigue-range. The Author.

At temperatures above 300° C., which were not of great interest o the bridge engineer but were of great interest theoretically, the yclical rate was of very great importance, and it would appear that vhen the static stress that might be applied indefinitely without onditions of creep approximated to the maximum stress at which racture took place quickly, then the cyclical rate had little effect n the fatigue-range. It was very doubtful also whether periods of est had any great effect; for example, in carrying out tests on naterials, it had often happened that during some of the tests of he series, the machines had been stopped for long or short periods,

The Author. but when fracture had occurred, points on the plotted curves corresponding to those delayed tests fell on the curve obtained from the tests run continuously. As Mr. Nichols suggested, all loadings on a bridge did not produce the maximum stresses. The effect of repeated loads below certain ranges did not appear to have any effect in producing fatigue in mild steels, and it appeared, therefore, that only those loadings producing stresses beyond the fatigue-ranges determined experimentally were of real importance. In that connexion, the shape of the plotted curves was of importance.

The point raised by Dr. Nichols as to the "effect, if any, occasional high stresses were likely to have on the ultimate endurance of a member subject to repetitions of stress just below the fatigue limit," was of very great importance. The occasional high stress might be sufficient either once or when repeated a few times, to start a crack, and as at such cracks concentrations of stresses would occur the cracks might continue at much lower stresses. The Author, in a recent Paper,[1] had shown that under corrosive conditions, for example, corrosive penetrations might occur which would finally lead to fracture at comparatively low stresses. He had not been able to show that stresses below the fatigue-range changed the percentage of elongation of mild steel. He had, however, been able to show that certain austenitic steels might be considerably hardened by the stress-cycles, but that that hardening accompanied by the increase of the tensile strength and a diminution of the percentage elongation might take place without any sign of failure under fatigue. Further, mild steels might be hardened by cycles which produced plastic deformation.

Mr. H. B. Gates pointed out, quite rightly, that the bearing stresses on the rivets on the specimens that failed at 8 or 9 tons per square inch was 13·33 tons per square inch. It should, however, be noticed that the ratio 13·33 to 9 was less than 10/6, which would probably be the design-ratio. Nevertheless, the Author was inclined to agree that the bearing stresses in that case might have contributed to the low fatigue-range. As Mr. Gates observed, the transference of stresses through riveted joints was a complicated matter and the Author hoped that much more experimental work would be undertaken.

It might be of interest to point out that in a Paper shortly to be published, it would be shown that failure at holes in flanges of rolled-steel joists, with rivets in them, took place at ranges of stress of about five-eighths of those at which the flange without holes failed.

[1] "The Effect of Discontinuities and Surface Conditions on Failure under Repeated Stress." *Engineering*, vol. cxliv (1937), pp. 87 and 140.

CORRESPONDENCE
ON PAPERS PUBLISHED IN
DECEMBER 1936 JOURNAL.

Paper 5060.[1]

"Ship-Canals Utilized for Drainage."

By LUDOLPH REINIER WENTHOLT, D. Tech. Sc., M. Inst. C.E.

Correspondence.

Mr. R. G. CLARK agreed with the Author that, generally speaking, Mr. Clark.
he interests of navigation and land-drainage operating in one inland vaterway did not always coincide. In high peak-flood periods, low vater-levels were necessary in the reservoir to pump land-drainage nto without stoppage or interruption. On the other hand, navigation required all the water obtainable, as it was easier for ship-ropulsion and steerage.

Conditions in Holland and England differed considerably, and arges with a capacity of up to 2,000 tons were used in Holland, s compared with those of about 30 tons capacity used in England. Again, the difference in tidal range in England and Holand was great; he believed that the tidal range in Rotterdam was bout 6 feet, whilst on the seaward side of the Zuider Zee embankment the range was probably only about 2 feet.

He agreed with the Author that the size of the craft should have minimum ratio of 7 to 1 to the wetted perimeter of the canal or vaterway; in other words, the larger the craft the larger should be he canal-capacity. That was also helpful to land-drainage, as it rovided a larger reservoir to accommodate the surplus flood-water rom the adjacent land. The ratio of the canal-capacity to the area rained in Holland was notable.

He agreed with the Author that the critical velocity in inland vaterways was governed by the material in the bed and in the sides f the canals. He had met with hard gravel, clay, and fine sand vithin a distance of 10 miles, and it was the velocity that eroded he fine sand that controlled the velocity of the canal. It would be of nterest if the Author could show in a diagram the hydraulic curves rom end to end of the canal during sluicing under flood conditions t Ymuiden. With neap tides and adverse winds the sluicing-range nd time-factor would add interest to the Paper.

[1] Journal Inst. C.E., vol. 4 (1936–37), p. 221 (December, 1936).

Mr. Clark. What was the effect of wash from the self-propelled craft? He had noticed in northern France that the wave following large craft was sometimes not so large as the wave following smaller craft, but the question of speed was a factor to be taken into consideration. It might be that the wave after a vessel passed caused more damage to the canal-sides than the velocity due to sluicing the excess water; alternatively, the combination of both might cause the maximum damage. He would be interested to learn the results of the Author's experience.

Mr. Gardner. Mr. A. C. GARDNER pointed out that in a country like Holland drainage considerations had of necessity to take first place, and so long as navigation interests were not hampered by the restricted size of vessels both purposes could be equally served. When, however, as in the case of the North Sea canal, provision had to be made for vessels with a draught of 9 metres, it was at once apparent that the problem to be faced by the engineers became one of the first magnitude, involving as it did questions other than those of actual construction (important though the latter were). The Author stressed the importance of the most suitable water-level in a canal to suit both conditions, drainage requiring a low water-level and navigation requiring a high one. The problem was further increased by the large quantities of water brought into the canal in rainy seasons, amounting to over 7,500,000 cubic metres per day, and further by the limited output of the pumping plant at the eastern end of the canal, and by the tidal conditions controlling sluicing-operations at its outlet into the North Sea at Ymuiden.

The Author stated that, as the result of reconstruction-works in hand, the cross-sectional area of the canal was being increased from 800 to 1,440 square metres, an increase of 80 per cent., but that he did not anticipate that the maximum velocity of flow would be greater than at present. It appeared that with the greatly increased carrying capacity of the canal the velocity would be materially reduced. The mean velocity of 0·5 metre per second did not seem to be very high for a canal of the size mentioned, where the ratio of the perimeter of the largest vessel using it to the perimeter of the canal itself was as low as 1 to 7. There were many navigation-canals where that ratio was much higher, and where the mean velocity was also greater. In the case of many canalized rivers such conditions might be regarded as extremely favourable. To cite only the case of the river Clyde, a tidal velocity of more than twice the figure referred to above was quite common, whilst the perimeter ratio was higher over considerable portions of the length of the navigation. In the extreme case of the *Queen Mary*, when that vessel was leaving the shipyard the critical ratio was about 1 to 4

That was naturally exceptional, but in many cases the ratio would be about 1 to 5 or 1 to 6. Would the Author give the ratio f/F in respect of the three canals referred to on p. 225 § ? Mr. Gardner.

The question of maximum permissible velocity was also a matter upon which he would like further information. He agreed that the velocity ought not to be so great as to render navigation of ordinary vessels difficult, and that it should be below the known critical velocity at which the material composing the banks of the canal would be transported.

The points considered above would appear to be the main desiderata, and it was not clear why traction-costs per horsepower-hour, which in their turn depended on the kind of power used and on variable fuel-prices, entered into the question of canal-design, unless the canal-proprietors were themselves the owners of the transport and were vitally concerned with the question of economical transportation. Perhaps the Author would be good enough to answer that question.

The Author stated that by enlarging the cross-section of the Twenthe canal over a length of 20 kilometres to one-and-a-half times that required for navigation, it had been possible not only to avoid the costly construction of a number of culverts under the canal, but at the same time to increase the value of adjacent lands by an amount equal to 75 per cent. of the total cost of the works. That being so, why was the straightforward expedient of enlarging the cross-section of other canals not more commonly resorted to, seeing that the permissible velocity increased with the increase of the ratio F/f and that the amount of drainage-water which could be discharged through a canal increased rapidly with any increase in its cross-sectional area ?

In the design for the inflow structure for the Bolksbeek the decision to place the settling-basin above the weir was undoubtedly wise, as was also the decision to place the weir at an angle to the canal, whilst the placing of the sill and floor out of level in order to secure the least disturbing inflow to the canal was a very ingenious method. It would be of interest to know if that method had been adopted elsewhere, or whether it had originated in the experiments to which the Author referred. What normal method of protection was adopted on the dykes in open stretches of the canal ?

The conditions governing the requirements of the spillway near Zutphen appeared to have been well considered in relation to the varying seasonal drainage-discharges and to the probable river-levels in the Yssel, whilst the experiments in the hydraulic laboratory

§ Page numbers so marked refer to the Paper. (Journal Inst. C.E., vol. 4 (1936–37) (December, 1936).)—Acting Sec. Inst. C.E.

Mr. Gardner. at Delft also appeared to have contributed useful information towards the detailed features of the design. Concrete seemed to have entered very largely into the construction-works described. To what extent, if any, were the spillway-face, sills, and culverts protected by stone facing of any kind?

Mr. Peel. Mr. Cecil Peel observed that many navigable waterways were tidal or non-tidal rivers which had been deepened and trained, and which therefore had certain currents along them, whilst even in canals considerable movement of the water was caused by the passage of vessels. All rivers had tributaries, and if they were canalized or deepened the problem of the entry of drainage-water would arise in its relation to navigation. Again, it was seldom possible to construct a canal, which necessarily had to cross a great many natural watercourses, without having to use it for drainage purposes.

Generally the water which flowed along a river or entered a canal from drainage-channels contained silt which was frequently deposited in the waterway, and could become a hindrance to navigation. A constant velocity which neither scoured nor allowed deposition represented the ideal, which was never attained in practice, in those types of channels. Usually the deposition was confined to a definite position in the channel, and a somewhat greater depth could be maintained there so as to have a reserve for silting. Large canals were usually constructed, and rivers deepened, to a trapezoidal cross-section, but it would seem that that was not a very favourable section from the consideration of silt-equilibrium. Natural earth-slopes, river-beds, and banks were never straight lines in cross-section.

It had been shown [1] that the natural shape of the cross-section of a straight channel through alluvium which was in silt-equilibrium was a semi-ellipse, and while that was seldom exactly the case in practice in a large natural channel, yet the section was always curvilinear.

Examination of the cross-sections of rivers and canals after they had been left undisturbed for some time, except by the action of currents and the passage of vessels, invariably disclosed a deposition of silt in the bottom angular corners of the trapezium, and frequently near the middle of the channel. In plan these were never absolutely regular, since a straight channel was not a natural one and the water tended to form a winding channel of greater depth between the shoals of silt. The reason for that typical

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[1] G. Lacey, "Stable Channels in Alluvium." Minutes of Proceedings Inst. C.E., vol. 229 (1929–30, Part 1), p. 259.

position appeared to lie in the existence of transverse currents in Mr. Peel.
e channel.

It had been shown [1] that in a straight channel along which water
s flowing, the level was slightly higher in the middle than at the
es, and that caused the water to have a helical motion. It tended
flow downwards in the middle, outwards across the bottom towards
e sides, up the sides, and inwards across the surface towards the
ddle again. In a channel having sharp angles between the bottom
d sides that flow caused eddies in the angles, and those caused
e deposition of silt, which might either be suspended in the water
derived from erosion of the bottom and sides. Again, the lateral
w was less in the middle of the bed and there slacker water and
position occurred. Thus the channel tended to form a curvilinear
semi-elliptical shape, and once formed there was a more uniform
eral flow, resulting in silt-equilibrium.

In constructing and maintaining channels, therefore, it was possible
at some benefit might result from making the section curvilinear
the bottom instead of entirely level, the sloping sides forming
ngents to the curve. That would quite possibly effect a reduction
the absolute amount of silting, which would be more uniform
ross the bed. Moreover, the silt would be deposited in a channel
ich was deeper in the middle than was actually required for
vigation, and so would interfere less with it. The cost of forming
e channel to that shape in the first instance might be considerable
more excavation would be required, but in a canal or river which
quired heavy maintenance dredging the extra cost would not
atively be large, and if economy in maintenance resulted, the
tra cost might be worth while. The curvilinear section would be
sy to form with machines of the land-dredger and drag-line types
rking across a canal, but with a floating dredger it would have to
formed in steps which would be transformed into a curvilinear
ape by the action of the water.

Another advantage that that form of channel would possess was
at, on occasion and by using special care in towing, somewhat
eper-draughted vessels could use the channel than would otherwise
possible. Again, there would be a greater depth in the middle
allow for the "squatting" of vessels when under way. That
uld amount to 6 inches or more in the case of a ship of 26 feet
ught navigating in a canal 28 or 30 feet deep, with a ratio of
tional areas of about $4\frac{1}{2}$. "Squatting" was due to the displace-
nt of water from ahead to the stern of the ship due to its forward

[1] A. H. Gibson, "Hydraulics and its Applications." Fourth Edition,
ndon, 1930, p. 331.

Mr. Peel. motion. That caused a current of water relative to the sides an bottom of the canal, the kinetic energy of which was derived part from a lowering of the water-level along the greater part of the ship length, and partly from the bow-wave.

The use of hydraulic models in the design of the various work described in the Paper was very interesting. The use of those model was nearly always justified in such cases as their cost was usuall small compared with the savings to be effected, and possibly ver costly mistakes might be avoided by their use.

No purely theoretical treatment of even the simplest of hydraul problems could be at all complete in the present state of knowledg and indeed quite erroneous calculations could easily be made. Th laws of similarity as applied to hydraulics were becoming bett known, and it was possible to construct models which gave corre results within the limits of the particular problems for which the had been designed.

Some years ago a model of the upper Mersey estuary had bee constructed and its régime had been compared with that of its prot type. That estuary possessed very marked features and quite larg changes took place relatively quickly, so that it provided an excellen example for comparison. Unfortunately the ratio of vertical an horizontal scales which had been chosen had not been as correct a it should have been, and subsequent research had disclosed the corre scales. After the model had been operated it was decided at the tim what scales would probably have given more exact results, and it interesting to note that they were the same that research had sinc shown would have been correct. Nevertheless, the correspondenc between the model and the estuary had been remarkable and a the main features had been faithfully reproduced in a general way With regard to the purely dynamic features of tidal-levels, current and relative times of high and low water at various parts of th estuary, the correspondence was almost exact.

In another model the object had been to determine the probabl extent of scouring at the outfalls of a large storm-water pumping station. In that case three pumps at times discharged up to 9 cusecs each through bellmouthed outlets of 51 inches maximum diameter. The outlets were fitted with balanced flaps at their end and were contained in recesses 9 feet 6 inches high and 7 feet wide in vertical quay-wall. The ground in front of the wall sloped along th wall, being about 3 feet below the sill of the recess in the shallowes case and about 12 feet below in the deepest case. The axes of th pipes were 3 feet $1\frac{1}{2}$ inches above the sills of the recesses ; L.W.O.S.T was 3 feet below, and H.W.O.S.T. 10 feet above, the sills, the max mum tide-level being 4 feet above H.W.O.S.T. When the water i

model discharged at L.W.O.S.T. the scour was very considerable. Mr. Peel.
the tidal level was raised the scour became much less, and when it
s raised to that of the axes of the pipes the scour ceased. For-
ately that was the lowest tidal level at which the pumps would
used in practice. Higher tidal levels were tried, and it was found
t when the water-level was a little above the H.W.O.S.T. the
ur recommenced slightly. That appeared to be due to the fact
t up to about H.W.O.S.T. a large amount of kinetic energy was
t in wave disturbance, but that above that level that disturbance
sed and the water seemed to strike downwards more directly
rards the ground. The model really showed that any scour that
ght take place in practice would not be serious and could be
erated, and such had been the case during some 5 years' operation.
was only by experiments with correctly designed models that
se peculiar effects could be discovered, and accurate hydraulic
asurements made. In the model of the pumping-station, which
luded the complete system of culverts, passages, multi-flap valve,
., the head-losses at different points for various flows were
asured.

With regard to the tests on turbulence-basins and their down-
eam sills, and the scour in downstream channels, research had been
ried out both in France and Germany upon that problem, using
dels as well as actual structures. Two systems had been evolved
ich, it was claimed, greatly reduced such scour and obviated the
essity for extensive aprons. In France the toothed sill had been
ented. That consisted of alternate higher and lower teeth,
tangular in section across the stream and triangular along it. The
her teeth generally had vertical upstream faces and sloping down-
eam ones, and the lower teeth had faces sloping in both directions.
e gaps between the higher teeth allowed some of the water to flow
o the eddies formed by the higher teeth and so reduced their
bulence and scouring effects. In Germany the grillage system
l been used in which timber baulks were laid with spaces between
m along the bed of the channel and slightly above it. Those
ilks sloped upwards slightly in a downstream direction, and a
ht gap was left between the end of the concrete apron and the
bers. The effect was that an eddy was formed and a reverse
rent set up underneath the grillage, and that carried in material
l so filled up the space and prevented scouring beneath the apron.
d the Author experimented with those devices in his model tests?
o, what kind of surface had he given to the model-walls and what
d material had he used in the models to test the scouring?

The Author. The Author, in reply to the Discussion and Correspondence,
nted out that in the Netherlands there were some large rivers,

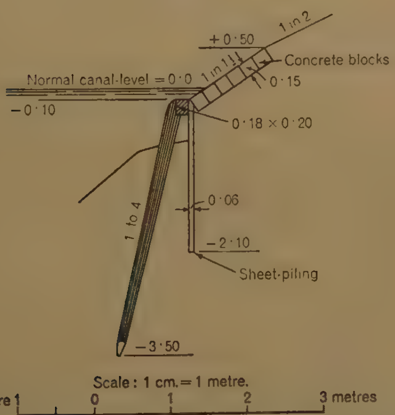
The Author. the Rhine and its branches and the Meuse, the last-mentioned havi been canalized. The Paper did not, however, deal with the Meu and some smaller rivers, which, notwithstanding their canalizatio had kept their character of rivers; it only referred to real cana independent of the question whether they had been in former tim natural waterways or not. The canals dealt with in the Paper we not under the influence of the tide, because of the fact that t waterways which had formerly been in open communication with t sea had, with the exception of the large rivers and the estuari been closed many centuries ago at their lower end by means of dam in order to prevent the high tides and the salt water from enteri into those canals. The dams were provided with sluices. Enginee in the Netherlands had, therefore, little or no trouble in connexi with silt carried by tidal water into the canals. Moreover, the sm rivers and brooks, which brought their water into the canals, general speaking had a catchment-area with a sandy soil, so that their wat contained little silt. On the other hand, those rivers and broo transported, especially during flood-conditions, a great quantity sand. To prevent that sand from entering into the canals, t inlets were provided with settling basins. As there was little or silt in the water and care was taken that the sand did not enter in the canals, dredging in the canals in the Netherlands was of sma importance. In consequence the fact mentioned by Mr. Peel, that trapezoidal cross-section of a channel was not a very favourab section from the consideration of silt-equilibrium, was not of pr dominant importance under the conditions which were met with the Netherlands.

The erosion of the side slopes due to wave-action might be great importance. It depended on the material of which the si slopes consisted, on the dimensions and the shape of the vessels, an above all, on their speed. The speed was therefore always limite The speed-limit was generally from 12 to 18 kilometres per hour f small vessels on large canals, and from 6 to 7·5 kilometres per ho for relatively large barges on the smaller canals. It had been show in practice that the erosion only occurred between from 0·5 1 metre below and and 0·5 metre above the water-level; the slop of the canals were therefore generally provided either with a pr tection of stone or mattress-work from 0·5 metre above norm high-water level to 1 metre below normal low-water level or wi a construction of steel or wood sheeting of such dimensions that kept its position even if the slope had been eroded down to 1 met below the water-level. The normal method of protection of t banks of the Twenthe canal was shown in *Fig. 9*. At places wh boats were often brought alongside the bank the influence of t

opellers was very bad; at such places the protection had to meet uch more severe conditions. The concrete blocks were hexagonal. ıey had a height of 15 centimetres, and the length of the sides ıs 12·5 centimetres. The concrete was of a composition of 1 part cement to 6 parts of sand and gravel. The blocks were made in ecial presses. The Author.

The engineers and ships' captains in the Netherlands generally eferred a section with an entirely level bottom to a curvilinear ction, especially for canals with much navigation. A canal with level bottom had the advantage that vessels might pass with ore clearance between them and between the vessel and the slopes ıan was the case with a canal with a curvilinear section of the same

Fig. 9.



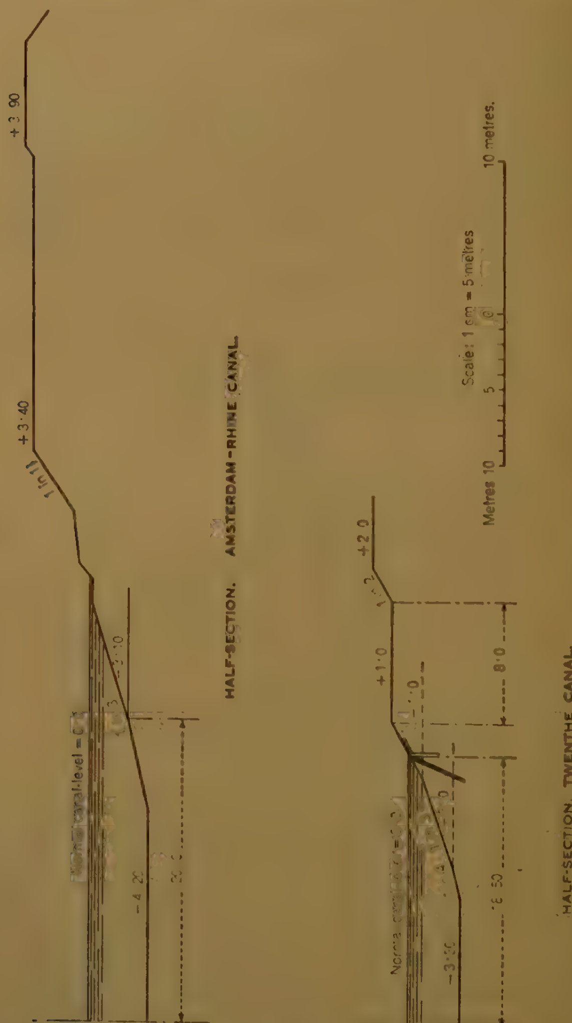
BANK-PROTECTION ON THE TWENTHE CANAL.

ːea. The normal section of the Twenthe canal and that of the new msterdam–Rhine canal (now under construction) were shown in *igs. 10*, p. 354.

The section would naturally not be constant; a deposition of ınd from the higher parts of the slopes would be found in the ɔttom angular corners of the trapezium, but Dutch engineers did ɔt think that a reason why that cross-section, which was in their ɔinion the best, should not be adopted. Moreover, the alteration ı section thus produced was of no great importance. Theoretically, ı advantage of a curvilinear form of canal was that vessels of ɔmewhat deeper draught could use the canal than would otherwise e possible. That advantage was not important in the Netherlands, nce the draught of the vessels was generally determined by the

The Author. depth of the lock-sills, and moreover because the depth of t channel was seldom less than 0·5 metre more than the draught the vessels for which the canal was designed.

Figs. 10.



SECTIONS OF AMSTERDAM-RHINE AND TWENTHE CANALS.

There were two reasons why the ratio of the perimeter of th largest vessel using a canal to the perimeter of the canal was perhap somewhat higher in the Netherlands than in many other countries In the first place there was little hard material in the bottom, an

the second place the contractors in Holland specialized in dredging, d Holland had a great fleet of dredgers. A larger cross-section erefore cost less in Holland than was the case in countries with her conditions. Moreover, there was an important traffic in the etherlands with rather small steam- and motor-vessels in regular rvice, for which a rather high speed was very important; hence followed that the cross-section was determined not by the dimenons of the very large, slow-going barges, but by the speed which as allowed for the smaller steam- and motor-vessels. Nevertheless ere were many canals, especially the smaller ones, for which the tio of cross-sections f/F was not very favourable. The Author.

He quite agreed with Mr. Gardner that a maximum velocity of 5 metre per second was not high for a canal of the size of the North a canal, where the ratio of the wetted perimeter of the largest ssel using it to the wetted perimeter of the canal itself was as low as to 7. The ratio f/F for the canal from Assen to Meppel was about : 4·2, for the canal from Zwolle to Almelo it was about 1 : 2·2, and r the South Willems canal it was about 1 : 4.

Mr. Gardner asked why traction-costs per horse-power-hour, hich in their turn depended on the kind of power used, and on riable fuel-prices, entered into the question of canal-design, less the canal-proprietors were themselves the owners of the ansport and were vitally concerned with the question of economical ansportation. As a matter of fact, inland navigation had almost ways to compete with rail and/or road transport, and the canaloprietors were therefore always concerned with the costs of transrtation. If those were low because the cross-section of the canal as large, they were able to fix higher duties than when the canal d a small section. With the same duties there would be more vigation on a large canal than on a small one. Mr. Wilson was terested in the assumption of a tug-pull of 1 kilogram per net ton, d asked how that pull was estimated. In Holland, and also in ermany, the opinion was that that pull was reasonable, and callations were therefore made on the assumption of that pull.

In most cases it was practically impossible to widen canals in the etherlands, because there were roads alongside them, and often uses were situated quite near to the canal. Apart from that it as generally very expensive to widen a canal, as enlarging entailed newal of the bank-protection, lengthening of the bridges and lverts, raising of dikes and roads, and so on. In the case of the wenthe canal the cross-section had not been enlarged when the nal was finished, but from the start its cross-section was one-and-half times that required for navigation. The ratio of the canalpacity to the area drained in Holland was not as notable as Mr.

The Author. Clark seemed to think. It was not the same for the different par of the Netherlands, but generally speaking it did not amount more than 4 per cent.

He quite agreed with what had been said by the President an others about the value of model-tests. Models generally cost onl a very small percentage of the actual cost of the works. The tria made in connexion with the weir of the Bolksbeek had cost 1,80 guilders (£200), and those in connexion with the spillway at Zutphe had cost 3,600 guilders (£400), less than 4 per cent. and 2 per cen of the costs of the works respectively. He appreciated the wor of Mr. Borer concerning Mr. Thijsse and his laboratory; M Thijsse and himself had worked together in very close co-operatio Generally speaking the models were being used to ascertain th relative merits of various designs, and not to make the design itsel In many cases, however, the designs were altered as a result of th laboratory-tests.

The placing of the sill of the weir of the Bolksbeek out of level i order to secure the least disturbing inflow to the canal had, as fa as Mr. Thijsse and he knew, not been adopted elsewhere. Tha method had originated in the experiments, referred to in the Pape An apron-length of 8 metres was sufficient because the water wa falling vertically and the discharge was divided over a rather larg distance, so that the maximum discharge per metre of sill was onl $1\frac{2}{3}$ cubic metre per second. The great height of the sill was necessar to get the flow well divided and well directed. A hydraulic jump di not take place. Nevertheless the water flowing over the sill necessi tated the provision of mattress-work on a scale similar to tha anticipated in the laboratory. The toothed sill mentioned by M Peel had not been invented in France, but by Professor Rehbock o Karlsruhe. The other construction (the grillage system) referre to was of Austrian origin. With both systems research had bee carried out at the laboratory at Delft, but they had not been though suitable for the Twenthe canal. The surfaces of the model-wall were of paraffin-wax, of cement-plaster or of painted wood. The were made as smooth, in proportion to the scale, as the actu works. Pumice sand had been used in the laboratory as a materi in the models to test the scouring action.

Paper No. 5048.[1]

"Efficiency Tests of Large Modern Pelton Wheels."

By ERIC NORMAN WEBB, D.S.O., M.C., M. Inst. C.E.

Correspondence.

Mr. HENRY HEADLAND, of Arapuni, New Zealand, observed that any novel and interesting features had been incorporated in the esign of the machines,[2] particularly in the way of foolproof protection; from the details given in the Paper those devices had been sed to advantage in making testing-conditions ideal. The hydraulic onditions existing at Shanan for the machines under consideration ave a specific speed of 5·12, which was ideal for the type of turbine stalled (namely, a single-jet Pelton wheel with a ratio of pitch ameter to jet diameter of 10. A check of the leading dimensions the machines showed that they complied with normal standard ractice, and it was satisfactory to know that the accepted rules design could be followed safely for machines of large output. Mr. Headland.

In machines of the type under consideration two items contributed ainly to the value of the final overall efficiency; namely, the design the buckets and the nozzle. In view of the high values of efficiency btained at all loads it would be useful if the Author would give a ttle more information concerning the design of those particular ems.

For an efficient machine the jet should not be so large that it interred with the back of the buckets, and the buckets themselves ould be so designed that the jet was gradually deflected by the ucket-curves through an angle of 180 degrees, and then freely discharged without striking the wheel. The surface of the bucket ould be perfectly smooth and of minimum area to reduce frictionss. Those requirements were to a certain extent conflicting, and sketch showing the leading dimensions of the buckets, including e width of out-cut and the values of the various angles, would be useful addition to the Paper. From the data given in the Paper was estimated that the lip of the buckets should be undercut at angle of 73 degrees if impact of buckets on the jet were to be oided. If a calculation were made for the number of buckets quired to develop the specified output it could readily be seen that e machines had been designed for maximum efficiency apart from

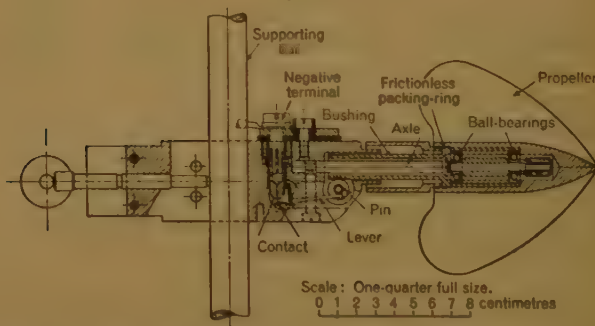
[1] Journal Inst. C.E., vol. 5 (1936–37), p. 259 (December, 1936).

[2] Additional information was published in *The Engineer*, vol. cliii (1932), 48.

Mr. Headland, other considerations. The minimum number of buckets requir was fifteen, so that there were actually about 50 per cent. more tha were actually required. Experience showed that within limits t efficiency of a given wheel increased with the number of bucket and apparently that was the reason for casting two buckets on t one lug so that all the buckets could be accommodated on the disc

It was stated in the introduction to the Paper that a special stainle steel was used for the buckets, presumably in anticipation of troub with corrosion at the backs of the buckets; after 2 years' operatio experience should have proved whether stainless steel had been effe tive in overcoming that trouble. In one case, ordinary cast bron steel, nickel steel and stainless steel had all proved so unsatisfacto under a head of 800 feet that the efficiency of the machine ha obviously been affected. An analysis of the special steel used for th

Fig. 12.



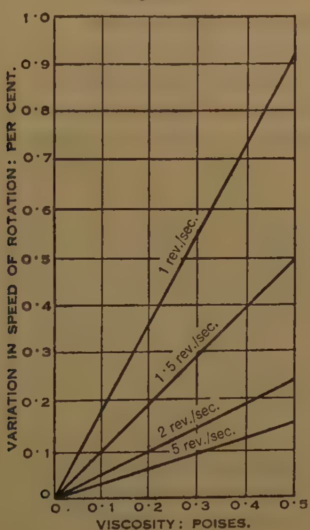
Shanan buckets would probably show that it contained about 14 p cent. chromium, 0·2 per cent. nickel, and 0·35 per cent. manganes with a Brinell hardness of approximately 200, and heat-treatme at 800° C.

Limitations of space had probably prevented the Author fro giving full particulars of the equipment used, and it was assum that the current-meters were the Texas type Mark V, a cross-secti of which was shown in *Fig. 12.* A short description of the constru tion and operation of the instrument might be of service to those w were not familiar with the apparatus.

The instrument consisted essentially of a two- or three-blad propeller, in the hub of which was mounted a pair of oil-immers ball bearings, the entrance of water to the chamber being prevent by a frictionless packing-ring. In the ball-races an axle was fitt which transmitted the rotation of the propeller through a bushi with a single or double thread and thence to a toothed wheel,

ch one or two pins were arranged, so that by combination of the Mr. Headland.
ead and pins signals could be obtained every 5, 10, or 20 revolutions required. The toothed wheel and pins operated a lever which ried a contact arm and contact, all of which were housed in a mber in the main body of the instrument. The contact and ative terminal provided for the opening and closing of a 4-volt al-circuit, which was completed through the meter supporting-. Instruments of that type were usually calibrated in still water, were particularly accurate; as pointed out by the Author, ocities could be obtained accurately to within 0·001 metre per ond. Apart from errors due to oblique currents, which would be

Fig. 13.



rred to later, the only inaccuracy which was liable to occur in ctice was that due to the viscosity of the oil used, which might rise to errors of up to 1 per cent. in the rate of rotation, the r increasing with the viscosity and decreasing with the speed otation, as shown in *Fig. 13.*

or velocities in excess of 1 foot per second the screw-type meter stered accurately near the walls of a pipe, but to get within 2·5 es of the wall either a 4-inch propeller was used, or the standard ch propeller was suitably shrouded, which would probably give to a slight error in the registration. A change of shape of the peller by deformation during operation might affect the rating re, and although no mention of that point was made in the

Mr. Headland. Paper, errors due to that cause could be detected by checking t shape of the screw against a plaster cast.

The Author had used two instruments in carrying out the tests although a large number would have resulted in a saving of tim and greater precision, because the results would have been le dependent on the existence of a steady flow. On the other han however, that would have involved the use of more adequate equi ment, including an electric chronograph for recording time an current-meter signals on a chart. It was assumed that the Auth had used a telephone or buzzer together with a stop-watch, and the speed of rotation would not exceed 7 revolutions per second maximum velocity that was probably satisfactory with contac signal at every 10 or 20 revolutions, although the liability of person error was not eliminated. The method of supporting the instru ments during tests did not seem very satisfactory, and it woul perhaps have been somewhat better to have used some rigid form of traversing gear with stream-lined rods, because high lateral stresse on light circular rods due to water flow might cause vibration of th mounting, which would result in the meter giving readings less tha the actual flow.

The minimum number of measuring points for a pipe having a internal area of A square feet was given by the relation $n = 4{\cdot}3\sqrt{A}$ which for the case under consideration gave 19, so that 21 points as provided, should be adequate. Judging from the polar diagram subsequently obtained it was possible that more accurate result would have been obtained if additional measurements had been take towards the outside of the pipe where the greatest valuations i flow and maximum velocity existed, instead of utilizing an equa distribution of measuring points along the diagonal axes of the pip

The magnitude and nature of the error in approximating the tot flow on the assumption that the velocities varied regularly as co centric circles was useful to know when making a rapid field-chec on turbine-efficiency. In connexion with the other two analytic methods of arriving at the areas of the polar figures, it should b pointed out that, while those approximations agreed, they usuall gave a somewhat higher result than the measurement of the actu area of the figures with a planimeter, in which case it might be possib that the values of Q obtained with the Ott meter might agree mo closely with those expected from the curve forecasting the je diameter d and the formula $Q = 0{\cdot}97(\pi d^2/4)\sqrt{2gH_c}$ (p. 271 § The known calibration of the jet based on experience and mode

§ Page numbers so marked refer to the Paper. (Journal Inst. C.E., vol (1936–37) (December, 1936).)—ACTING SEC. INST. C.E.

ts had served as a very useful guide to the accuracy of the Ott Mr. Headland.
ter gaugings, and such a substantial check was unfortunately not ually available in efficiency-tests on other types of turbines.

A comparison of the results of the tests between the machines th clean and lime-encrusted buckets showed a possible loss of iciency of up to 1 per cent., and illustrated how important it was maintain the buckets in good condition.

The flow of water in the pipes as illustrated by the polar diagrams r No. 1 and No. 4 machines were such that the efficiency figures tained might be expected to be optimistic. As pointed out by the thor, the conditions were similar to those obtained in the Arapuni rbine-tests (p. 280 §). In the latter case it was probable that the sturbance produced at the intake, which was practically at right gles to the flow in the headrace, together with the subsequent wnward bend in the penstock, continued down the full length of e pipe. Such a disturbance might persist for more than thirty ameters, which in the case of the 12-foot diameter Arapuni pen-ocks would give a length of 360 feet, the approximate overall ngth being 450 feet. The location of the measuring instruments uld, however, bring them into the zone of the disturbance produced the intake. At Shanan, the Author stated that the diameter of e pipe was sufficiently small to provide retardation to any rotational ovement which would eventually be damped out in a sufficiently ng stretch of straight pipe, but the bifurcation, as well as the rtical and horizontal bends, together with the Johnson and tterfly valves in the pipe, would also bring the current-meters into e region where the disturbance produced by those items still isted.

As noted in paragraph (*c*) of the conclusions (p. 279 §), the existence spiral flow would have an appreciable effect on the value of the locity obtained, for, since the instruments were calibrated in still ter, oblique or turbulent flow introduced certain errors which luced the reliability of current-meter measurements. If the rbulence were not too pronounced the use of two instruments with ferent registration coefficients might be used as an index to the ount of correction to be applied to the more accurate of the two ters.

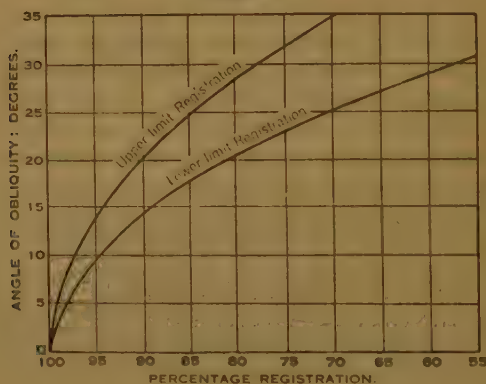
The Author had not indicated whether a two-bladed conical screw the three-bladed cylindrical screw had been used during the tests. hilst the former was self-cleaning, an advantage with lime-bearing ter at Shanan, it gave less accurate results where oblique currents re involved. The shape of the screw, however, was not the most

§ *Ibid.*

Mr. Headland. important feature, and where steady oblique currents were known t exist it was advisable to allow the axis of the meter to swing in li with the flow and to make an accurate observation of the ang rather than to operate the meter crosswise to the current; the latt resulted in the propeller being struck at an oblique angle by strea whose velocity varied rapidly in magnitude and direction, and a involved a non-uniform distribution of the velocity-elements ov the plane of the blades. In consequence current-meters definite under-registered the forward components of oblique lateral a vertical currents, which were to a certain extent influenced by inte ference of the meter-supports.

In *Fig. 14* the range of possible error at the maximum veloci

Fig. 14.

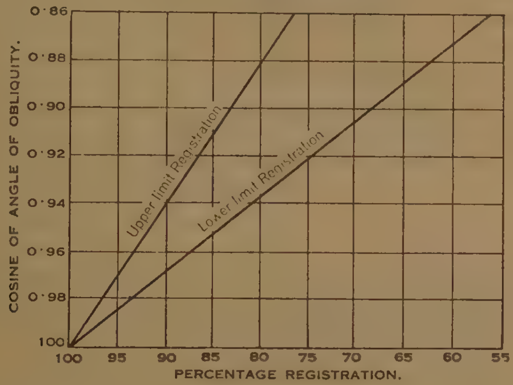


in the pipe was shown plotted against the angle of obliquity, whil in *Fig. 15* those curves were shown with the cosine of the angle ordinates. They were straight lines, indicating that the mete integrated correctly all cosine-components, independently of vari tions in the obliquity.

The unreliability of pitot-tube measurements under flow-conditio experienced in large pipes was now well recognized, but recently instrument known as the "pitot-sphere" had been developed f which was claimed freedom from errors due to obliquity and tu bulence. The measurement consisted of a comparison of the pressu distribution at five points on the surface of a 12-millimetre diamet sphere, from which the direction of flow, velocity, and static pressu could be calculated. The method had been used for pipes up to 6 millimetres in diameter, with satisfactory results compared w other methods of measurement. The instrument had the advanta

being simple to operate and inexpensive, and there seemed to be reason why it should not be used with pipes of larger diameter. Mr. Headland.

Fig. 15.



The AUTHOR, in reply, observed that it was hardly practicable or rmissible to go into details of the bucket design, concerning which r. Headland's comments were generally correct. Buckets of ainless steel had been adopted to obtain (*a*) great resistance to the ual wearing and corroding influences, and therefore sustained high iciency over long periods, and (*b*) increased resistance to any cipient cavitation or cognate effects. After 4 years' operation commercial load, it had been reported that the bucket-tips and litting edges were as sharp as when installed, and that only the ghtest signs of wear of any kind had been discernible anywhere, ilst the original polish had remained undiminished. The Author.

The current-meters at Shanan had been of the two-bladed conical-opeller type. It was believed that the rod carrying the propellers s amply stiff to prevent any vibration at the velocities measured; rtainly no vibration of the rod could be detected by usual physical eans. It was doubtful whether any increase in the number of easuring points within the area already covered would have ected the results. Additional points in the outermost annulus d next the skin, however, were most desirable, and would be luable for checking the somewhat arbitrary value of the velocity cepted for the flow next the skin. The only practicable means for ecting that would appear to be the pitot-tube in one of its later and re reliable forms. The tendency to spiral flow at the metering tion could not have been other than slight, and the angularity rrespondingly small. Without some other equally reliable form check-measurement, it was not possible to estimate what effect

The Author. such angularity as actually existed might have had on the accurac
of the velocity measurement. A reliable pitot-tube velocity-met
would be a great boon, and there seemed reason to expect that
become a practical reality of the near future. The Author had ha
no experience of the "pitot-sphere" referred to by Mr. Headland.

CORRESPONDENCE
ON PAPERS PUBLISHED IN
JANUARY 1937 JOURNAL.

Papers Nos. 5054 and 5055.[1]

"The Lower Zambezi Bridge."

By FREDERICK WILLIAM ADOLPH HANDMAN, C.B.E., M. Inst. C.E.

and

"The Construction of the Lower Zambezi Bridge."

By GEORGE ERIC HOWORTH, M.C., B.Sc., M. Inst. C.E.

Correspondence.

Mr. W. J. DOAK, of Brisbane, considered that further information Mr. Doak. ight have been given in regard to the depths at which the bridge-iers should be founded. Mr. Handman, for instance, after describing ow careful investigation had been made as to the greatest probable epth of scour, stated on p. 334 § that it was assumed that 60 feet as about the maximum depth, and that, assuming 50 feet below ed-level would be safe, it was considered that a depth of 110 feet elow low-water level would be sufficient. Surely something more ientific than such a simple assumption was involved in the deter-ination of the founding depth ? Literature on the subject of ier-design was meagre and unsatisfactory, and as the reduction of erhaps 20 feet in the depth of all the piers in the bridge would have ved many thousands of pounds, it was remarkable that the science pier-design had made so little progress.

Experience in sinking that class of pier had established that the rface-friction of sand against concrete was much the same at all epths, and had a value of about 4 cwts. per square foot. That being e case, Rankine's theory of pressures in granular materials would ve to be abandoned, on the reasonable assumption that the efficient of friction remained constant. If that were true, how ould the theoretical bearing power of the sand be affected ? Since crease of depth apparently did not increase lateral pressures, the sistance to upheaval might not increase either, so that it was not parent what was gained by going to great depths. For instance,

[1] Journal Inst. C.E., vol. 4 (1936–37), pp. 325 and 369 (January, 1937).

§ Page numbers so marked refer to the Papers.—ACTING SEC. INST. C.E.

Mr. Doak.

it might have been as safe to found at, for example, 80 feet belo low-water level, because the bearing power of the sand was just a great at that depth as at 110 feet, and the load on it would have bee reduced by about the same amount as the frictional support. Whil considering the question of foundations, what class of material wa met with in piers 23, 25, and 26, which Mr. Howorth said wa unacceptable (p. 398 §)? Mr. Howorth's views upon the desig of the piers were of great interest, and it appeared that different type of well altogether might have saved much time an trouble. For instance, a circular well 30 feet in diameter, with a outer wall 12 inches thick, would be an attractive design; the nece sary weight for sinking could be given by a solid cylinder 10 fe diameter in the centre, connected to the skin by radial walls, thu giving a number of dredging wells close to the outer skin. The cent cylinder would have a sharp conical point at the bottom, forcin the sand into the wells. When the necessary depth was reache and the bottom sealed, the wells might be filled with water rathe than with sand, and the open top then filled to such a depth wit concrete as was necessary for supporting the piers above.

Mr. Fereday.

Mr. H. J. Fereday could confirm that the geological and physic conditions in the area, as ably described by Mr. Handman, had calle for anxious preliminary study in deciding upon the location of th bridge, and also that the foundations had been carried down to s great a depth not only as a very necessary precaution against scou but also by reason of the risk of earthquakes. Although the tremo experienced during construction had been of no great intensit stronger movements might occur at any time.

The allowances made in design for wind-pressure as well as f impact were in accordance with requirements of the British Standa Specification. In point of fact, however, the wind-pressure was n a controlling factor in the design of the bridge-members.

With regard to equating the cost of the superstructure with that the piers and establishing the length of unit span on that basi since such a computation had necessarily to be made upon pr liminary estimates of cost, the equation was seldom likely to b exactly fulfilled when actual contract prices were applied. Accordin to the original estimates the span-length adopted was the econom length. Mr. Nichols had suggested in the Discussion that the su members to halve the effective length of the posts were unnecessar and that they were a source of secondary stress. Their omissi would have necessitated heavier posts, and their contribution secondary stress was of little consequence.

He also wished to emphasize the great technical interest taken

§ *Ibid.*

e fungus attack and bacterial action upon the paint. That experi- Mr. Fereday. ice indicated the desirability of experimental observation to deter-ine the suitability under site-conditions of any paint proposed for important work in a tropical climate.

It was a source of satisfaction that, by the means provided, cluding the special domes for compressed-air work, and by the ergy and resource of the men on the spot, such difficulties as ose had been successfully overcome.

Sir ROBERT GALES observed that the final design of the bridge Sir Robert Gales. rmed a happy illustration of the value of collaboration between e two firms of consulting engineers.

The method described in Mr. Handman's Paper for determining e foundation-depths of the main piers had been found satisfactory r alluvial rivers in India. He was unable to agree with the remarks Mr. Handman's Paper under the heading of skin-friction. So far om a foundation of sand not generally offering the resistance ecessary to carry the full load safely without the assistance of skin-iction, his experience at many bridges was that sand under piers hich was or which had been already compressed by overlying sand nd water, and which was so enclosed that it could not escape terally, was practically incompressible and formed the best of all undations. Moreover, since in spite of all precautions cases did ccur in which the sand around the pier was scoured away below the sumed minimum bed-level, no credit should be taken for skin-iction in the design of bridge-well foundations.

Mr. Handman introduced a term "critical depth" which he efined as the depth "when a well has been sunk as far as it will without loading kentledge and without pumping," and sought to educe from it values for skin-friction. That critical depth was a mewhat indefinite point because if the well were plumb and the abs were being worked simultaneously, the well would continue go down for some time at the cost of an increasing excess of aterial dredged over the content of the well sunk. That showed the formation of a crater around the well at ground-level. Clearly figure for skin-friction could be obtained under conditions in hich the sand was going down with the well. It was, moreover, oubtful whether in the course of sinking in sand the cutting-edge nd cant-plate of the curb offered any resistance to sinking at the oment of movement, since, when a well was being steadily dredged, e water in the well was lower than the water outside, with the sult that the exterior surface of the well was being lubricated and e sand under the cutting-edge being softened by a steady though ght flow of water, such as took place in a greater degree when the side water was rapidly pumped out.

It was possible that an approximate figure for skin-friction might be

Sir Robert Gales.

obtained at the depth at which the volume dredged began to excee the content of the well sunk, but that had not yet been done. Th figures, expressed in cwts. per square foot as was customary, woul represent the average value of skin-friction from ground-level to th depth of the cutting-edge. In the present state of knowledge very littl use could be made of skin-friction in the design of well-foundation

By the sinking-effort method, however, well-foundations could b designed with considerable accuracy to be of a weight suitable fo sinking in sand to any required depth by dredging and pumpin without additional weighting. As some difficulty appeared to b felt in understanding that method as set out in the Paper on th Curzon bridge,[1] referred to by Mr. Handman on p. 334 §, the oppor tunity was taken to try to make it clearer. The sinking-effort pe square foot of the exterior surface of a well was calculated by dividin the weight in water of a linear foot of the well-steining, expressed i cwts., by the area of the external surface of the slice of steining i square feet. The main principle of the method was the preparatio of a depth–sinking-effort diagram plotted from cases in which actua wells of known weight and dimensions had been sunk to known depth below ground-level in sand by dredging and pumping, but withou added weight. To design a well all that was necessary was to dra out the plan of a well with normal dredging holes and of sufficien size to take the base of the pier with a suitable margin and, assumin the material most readily available, to calculate the sinking-effor which the design would furnish. If that sinking-effort were applie to the diagram it would show the depth to which the well could b sunk without added weight. If it were required to sink deeper i would be necessary to use a heavier material or to increase the thick ness of the steining, or both, until the design showed a sinkin effort which would be sufficient for the depth desired. It might b that that depth could not be reached with a practicable design c well, and the amount of added weight required to sink the last length could then be calculated.

The diagram given in Sir Robert's Paper on the Hardinge bridge, also referred to by Mr. Handman, had been confused by too muc information about the sinking at that bridge, and a revised diagram to which the sinking-effort of the Zambezi well had been added, wa shown in *Fig. 1*.

Mr. Howorth devoted much attention to the difficulties of sinkin wells by open dredging through thick beds of hard clay and sem

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[1] R. R. Gales, "The Curzon Bridge at Allahabad." Minutes of Proceedin Inst. C.E., vol. clxxiv (1907–8, Part IV), p. 27.

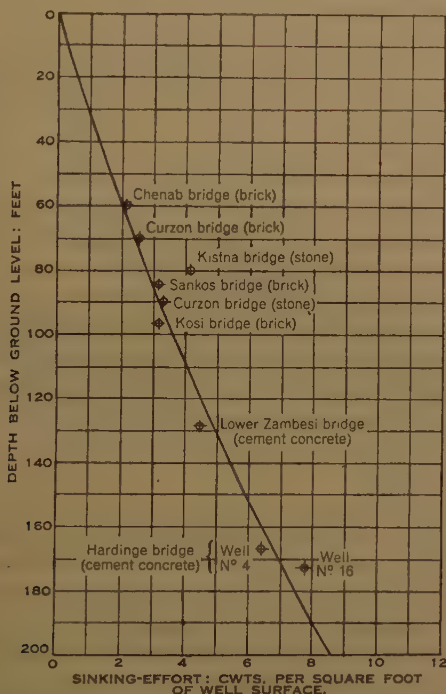
§ *Ibid.*

[2] "The Hardinge Bridge over the Lower Ganges at Sara." Minutes Proceedings Inst. C.E., vol. ccv (1917–18, Part I), p. 33.

ndstone, and criticized the design of the well-curbs with which he ad been supplied and the height above low-water level at which the er construction had to be commenced. Sir Robert Gales.

Sir Robert had had a very similar experience 33 years ago in sinking ouble-octagonal wells with curbs designed on the principles advoted by Mr. Howorth, as he had described in his Paper on the urzon bridge,[1] and he had come to the conclusion that no well could

Fig. 1.



Sinking-Effort Diagram for Bridge-Foundations Sunk in Sand.

(Hardinge bridge well No. 4 sunk by dredging only, and well No. 16 sunk by dredging and pumping.)

sunk through such strata by open dredging with anything like the me facility and certainty as by the pneumatic process, within the nit of depth to which that could be used. Very little advance d been made in sinking through hard strata by open dredging, but had been made possible by Mr. Fereday's ingenious domennexion, referred to in Mr. Handman's paper, to apply the eumatic process at any time to a well primarily designed for open edging, and all the wells of the Lower Zambezi bridge had been

[1] *Loc. cit.*

Sir Robert Gales.

fitted with the rings for that connexion. Great advances ha however, been made in the application of pneumatic pressure i foundation work, and by means of the decompression-chamber th depth to which it could be employed with safety had been muc increased. Mr. Howorth had placed on record that he had used pressure of 50 lbs. per square inch and over, and that the maximu incidence of " bends " for both Europeans and natives had occurre at a pressure of about 35 lbs. per square inch, and was about doub the mean incidence in both cases. A pressure of 50 lbs. per squa inch was equal to the pressure of 115 feet of water.

Examination of the sinking-diagrams of the main-span wel showed that, with the assistance of kentledge, progress through th upper black clay and the brown sandy clay had been well maintaine by open dredging, but that well No. 4, with a surcharge of 530 ton came to a stop at 104·5 feet below water-level after a month of ver slow sinking and was there founded, that well No. 21, with 688 ton was sunk into black clay and satisfactorily founded at 114 feet belo water-level without much difficulty, and that well No. 24, wit 1,000 tons, was sunk without much delay (except for time taken i loading the kentledge), to 107 feet below water-level and was then founded. It was evident that both Nos. 4 and 24 came to a sto because the grabs were unable to excavate the underlying strata In both those wells the conditions required the application of pneu matic sinking when the cutting-edge had reached a depth of 85 fee below low-water level.

Mr. Howorth had apparently not realized how extraordinaril easy the sinking of those bridge-foundations had been made for hin by the provision of sufficient weight in the wells themselves to ensur sinking in sand to the required depth by open dredging; by th provision of kentledge to assist in passing through strata, such a plastic clays, more difficult than sand but still dredgable; by th provision of the alternative method of penumatic sinking for strat too hard to be easily dredged; and, finally, by a curb with a shor and stiff plate-and-angle-iron cutting-edge capable of resistin deformation.

Mr. Howorth criticized the requirement that the tops of the well should be sunk to low-water level and that the piers should be starte from that level, but he made no definite suggestion for the case of th Zambezi river. He asked for " a working freeboard when foundin above at least the highest expected river-level during the workin season." What was a " working freeboard," and what was th length of the working season? The working season for the purpos of founding wells and starting construction of piers in tha river was very short, but it was not too much to expect of th construction-engineer that, with well-sinking made easy, he shoul

range his progress-diagrams to bring the founding of the wells and ers within the short working season. The tops of the wells could en be kept dry, if in a sandbank by a little pumping, and if in the ver by a removable cofferdam clamped around the well or by a mporary wall built at the edge of the top of the well. Alternatively, rmission had been given in the specification to build a short length pier, with dredging holes, on the top of the well, since deep wells ere not likely to alter in position during the last few feet of sinking. From the earliest times the neat work of the visible superstructure ad been set out on the rougher work of the hidden foundations at ound-level, which was represented in a bridge by low-water level, ad nothing was more unsighly than piers founded on wells which ere even only slightly out of position, all errors appearing at least be doubled. If, however, that were overlooked and the tops of ells were fixed at, for example, 5 feet above low-water level, very tle advantage would be gained. The working season would be tended for that work from a length of from 4 to 7 months to a length from 7 to 8 months, but the necessity would still remain to ascertain borings the depth at which the well would be founded, in order to termine the length of steining to be built to bring the tops of the ells to the same level; there was no one, least of all the owner, who ould tolerate well-heads of different heights, leaning in different direcons, with piers to all appearance perched precariously upon them. Sir Robert Gales.

Mr. H. B. GATES, of Perth, Western Australia, asked why the Pratt pe of truss had been employed, when the Warren type was demonrably more economical, simpler to design, and freer from secondary resses. That was now recognized by the American designer, r. Waddell, and it had apparently been recognized by Sir Benjamin ıker when he had designed the Forth bridge. Indeed, it was fficult to know why the Pratt truss had ever come into vogue, pecially the form with redundant bracing. It was also pointed t by Dr. Waddell that the correct measure of economical spangth was to make the cost of main trusses plus wind-bracing equal the cost of supports, omitting the cost of decking, etc., which was onstant quantity. However, in the case of a single-track bridge, e width between trusses had a limiting effect. Mr. Gates.

Mr. P. J. RISDON observed that Mr. Handman attributed the luction in the resistance of the smaller wells to sinking to the ghtly steeper angle of the cone-plate with the vertical. Mr. sdon had been largely concerned with the design of those wellrbs, and the following comments might be of interest. The area caisson having been determined by the permissible foundationessure, it remained to design the wells to offer the least resistance ithin reasonable limits) to sinking. When the design considered st had been evolved, a slight change had been made in the diameter Mr. Risdon.

Mr. Risdon. of the well, so that the air-lock domes used for the bigger wells cou be again employed for the smaller wells, thus saving time and expens Owing to the smaller size of well, it would have been difficult to app anything like the same weight of kentledge which the bigger we were capable of carrying, and that was an additional reason f reducing resistance to sinking. The weight of well being necessari much less, the best possible compromise between thick and com paratively thin steining had been arrived at, the chief aim havi been to reduce the clogging of the sand and clay at the junctions the cross-wall with the sides, by enabling the grabs to excavate near those points as possible. It was that, far more than the steep angle of cone-plate, that had accounted for the greater ease of sinkin It was true that the angle from the top of the cone-plate to the poin where the cross-cutting-edge joined the sides was also steeper, an that that was partly the result of a steeper angle of cone-plate, b Mr. Handman's statement might give rise to an incorrect impressi that a steeper angle of cone-plate in any type of curb would make the difference in sinking, which was far from being the case.

Mr. Howorth suggested that cross-cutting-edges could be omitte the possibility of that had been duly taken into account, but th had been deemed necessary for the sake of general rigidity owing the length of straight sides, and the small height of curb in pr portion to its length.

A type of curb that Mr. Risdon had designed 30 years ago might of interest. Ten curbs had been employed, each carrying an 8-fo diameter cylinder filled with concrete. The curbs had had to be sun through a maximum depth of water of 35 feet, and then throu about 20 feet of earth, down to a rock foundation where air-loc had been employed. The comparatively small size of curb (16 fe by 10 feet) with rounded ends, and the number of rigid permane upper strakes (skin-plates), had enabled a cross-cutting-edge to dispensed with, and the lowest cross stiffeners had been 6 feet 6 inch above the cutting-edge. The upper cylinders had limited the sing dredging hole to a diameter of 7 feet, but there had been no difficul in dredging, the earth having been free from wedging or "arching effect. A design for the smaller Zambezi curbs on similar lines h been considered, but such a design would have weighed and co twice as much as those actually provided.

Mr. Handman. Mr. Handman, in reply to the Discussion and Corresponden regretted that Professor Inglis had at first been puzzled by the u of the term "sinking-effort" in connexion with the sinking of t bridge-wells. That term had been used by Sir Robert Gales as f back as 1908 in his Paper on the Curzon Bridge,[1] and although

[1] Minutes of Proceedings Inst. C.E., vol. clxxiv (1907–8, Part IV), p. 23.

ıd not been in common use at the time, Mr. Handman believed that Mr. Handman.
had since become so. In addition to the explanation given by Mr.
oworth on p. 399 §, Sir Robert further defined its meaning in the orrespondence (p. 368) and emphasized the importance of the nking-effort method in connexion with the economical design of ell-foundations for bridges.

Professor Inglis reduced the sinking-effort graphs of *Fig. 16* . 342 §) to terms of the height of the well protruding above ground-vel at the various stages of sinking, which was correct provided at the varying water-level as affecting the weight of the well was so taken into consideration. He thought it would have been elpful if those graphs had given some information about the average nking-effort and sinking-resistances, but the graphs in question ere intended to demonstrate only the various stages of sinking the ells and the final depth to which bridge-wells of a given design ould be sunk entirely in sand without the addition of artificial eight or by pumping; in Mr. Handman's opinion the addition of ıy further details would have complicated the issue. Typical ıses of the sinking-effort of wells which were sunk in sand and strata ' harder material were shown by graphs in *Fig. 18* (p. 345 §) and ference was made to the nature of the material. The sinking-fort at any depth was shown by both the graphs mentioned above, ıt an average sinking-effort would be of little value for the reason ıat it was not governed by a linear relation but varied with the ater-level and with the varying preponderance of the weight of ıe well over the resistances during sinking operations, and only ecame a definite constant at or near foundation-level.

The measure of resistance to sinking by skin-friction was a con-oversial matter, and, as Sir Robert Gales pointed out, the present ate of knowledge permitted but very little use of it in the design ' well-foundations. The large number of wells of the Lower Zambezi idge which were sunk entirely in sand presented, however, an pportunity of adding to the information available on the subject ıd of an attempt at arriving at a closely approximate coefficient. r. Handman agreed with Sir Robert that during dredging opera-ons disturbances were created beneath the cutting-edge and cant-ate of the well by the steady but slight influx of water and running nd, and that lubrication took place on the exterior surface of the well. , however, the condition of the well were considered when at rest ıring the intervals of the dredging operations, and more particularly foundation-level, it was clear that further sinking would be pre-nted by two causes, (*a*) the resistance by skin-friction on the terior surface of the well, and (*b*) the resistance beneath the cutting-

§ *Ibid.*

Mr. Handman. edge and cant-plate, and the latter could not be ignored in arrivin at the resistance by skin-friction alone. When the well came t rest at foundation-level the sand which had been disturbed was no fully compacted, but it became so as the weight of the plugs and th pier was added, and the well settled down on a firm foundation. I the figure of 6 tons per square foot that he had assumed fairl represented the measure of the resistance of the slightly disturbe sand acting beneath the cutting-edge and cant-plate when the we came to rest, then 3·76 cwts. per square foot represented the measur of resistance by skin-friction. Sir Robert Gales further observe that his experience at many bridges had been that sand under pier which had already been compressed by overlying strata and wate and which was so enclosed that it would not escape laterally, forme the best of all foundations, but he added that in spite of all pre cautions cases did occur in which the sand around the piers wa scoured away below the assumed minimum bed-level. As no allow ance had been made in the design of the Lower Zambezi bridge well for the resistance by skin-friction, the actual position was that a the estimated depth of scour, 1·46 ton per square foot, or nearl 17 per cent. of the calculated unit load on the foundations of 8·17 ton per square foot, was taken up by the resistance by skin-friction Should deeper scour take place, thus releasing part of the latera pressure, there would be a margin of safety, but, of course, a diminish ing one.

Professor Inglis thought that it would have been valuable if mor observations had been taken to arrive at the measure of skin-frictio on wells which were sunk by compressed air. Mr. Handman ha taken a number of observations of wells which had been sunk partl in sand and partly in soft clay by open dredging and then by com pressed air, but the information obtained had been so inconclusiv as to be of little or no value ; the reason was that the excavation i the air-chamber had been carried beyond the cutting-edge of th well-curb, and consequently the space outside the skin-plate ha remained clear or had become partly filled with loose sand or othe material, so that either there had been no resistance by skin-frictio on that part of the well which was sunk by compressed air, or onl partial resistance corresponding to the amount of loose materia which had fallen into the clear space.

Sir Robert Gales had expressed the opinion that when a well sun entirely in sand reached what Mr. Handman had described as it critical depth, it would still continue to sink without the additio of kentledge or pumping if the well were plumb and the grabs con tinued to work simultaneously. It was evident that when the well approached the specified founding level they became sluggish, an that the resistances were beginning to take full effect. In the cas

f well No. 11, referred to on p. 343 § as being the well which had Mr. Handman. een sunk to the greatest critical depth, it would have been necessary o resort to pumping to sink it to any appreciable greater depth, and he fact that it had done so completely all that it had been expected o do was a tribute to its design.

He knew of no scientific method of determining with precision he greatest depth of scour which might occur in the bed of a river uch as the Zambezi, and he agreed with Mr. Doak that little progress ad been made in that respect; apart, however, from that indeter-ninate feature, it would seem that a distinct advance had been made n well-design, although he ventured to agree with Professor Inglis nd Mr. Nichols that it had not yet reached finality. There would ave been no justification for fixing the founding-level at a lesser epth than 110 feet, such as at the 80 feet suggested by Mr. Doak, or the reason that although it was known that the river-bed had een scoured to a depth of 60 feet below low-water level, it was not nown, and there were no means of ascertaining, to what depth t might be scoured at the site of the bridge when the river-flow vas obstructed by the piers. It was thought, therefore, that ample nargin should be allowed and that the security of the bridge should e further assured by placing around the piers pitching stones of uch a size that they would not be carried downstream when the iver was in flood. Moreover, the probability of earthquake shocks ad to be taken into consideration.

The conditions did not call for a circular well with multiple redging-holes such as was suggested by Mr. Doak, apart from the uestion of cost and speed of work. A 20-foot diameter circular vell with a single dredging-hole had been considered, but that lan had been dismissed for the reason that it would have necessi-ated the pier oversailing the well, which would have been par-cularly undesirable in view of the probability of earthquake shocks. Ir. Nichols stated that such a well was the easiest to deal with in nking, but it was questionable whether it would have been easier o control, and easier to rectify if it had gone out of plumb, than the ouble D-shaped well adopted. The speed and accuracy with which hose wells were sunk were a tribute to good work by the con-actors, and also a proof that they were controlled with great cility and that the design was a well-balanced one. That was still ore pronounced in the case of the seven smaller secondary wells hich were designed subsequently. It was calculated that they would roduce a sinking-effort 33 per cent. less than that of the main ells and that it would be necessary to load them heavily with entledge. Such, however, was not the case. They were sunk with

§ *Ibid.*

Mr. Handman. the greatest ease in sand and sandy clay by open dredging witho any kentledge or pumping until they reached soft sandstone, which they were sunk to the required founding-depths by compress air. Mr. Risdon was of the opinion that the steeper angle of t cone-plate of the curb of those wells would not account entirely the greater ease with which they were sunk, and with that M Handman was quite in agreement, as would be seen from the follo ing paragraph, which had been omitted from the Paper owing lack of space :—

> "The Author attributes the greater facility with which t secondary wells were sunk as compared with the main wel partly to the grab being able to dredge closer to the cuttin edge of the well-curb owing to the reduced thickness of t steining, and partly to the sharper angle of the cone-plates the well-curb."

The proportions of the piers and wells had been settled by co sideration of the dimensions necessary to accommodate the bearin supporting the spans and the bending stresses which might be pr duced by longitudinal forces acting at the bearings on the tops the piers. The dimensions of the wells had also been arranged facilitate sinking and were not controlled by consideration of th live-load figures, which, as Mr. Nichols stated, were very low wh expressed in terms of bearing pressure on the base of the well.

In order to make good the loss of cement when the concrete w placed in water, 25 per cent. was added to the cement-content the 1 : 3 : 6 concrete for the bottom plugs of the wells. He was agreement with Mr. Nichols that test-specimens would be mo representative of the concrete in a pier if they were cured under th same conditions of the pier itself, but it had not been found practi able to leave the specimens on a pier which was under constructio when a pier was being constructed on a dry sandbank they we buried in damp sand near the pier, or, in cases where the pier w surrounded by water, in damp sand on a barge moored to the pie for 24 hours (in both cases) before being removed to the testin room. The concrete caps to the piers, when finished, were ke moist for 10 days.

The bridge had been designed for impact, as mentioned by M Fereday, in accordance with the British Standard Specificatio but the Pencoyd formula had since been discarded in favour of new formula which permitted lower impact-coefficients, particular for the longer lengths. Professor Inglis asked if the total impac allowance was the very high figure of 284 tons, but that was not s as the impact-load of 71 tons on one bearing corresponded to th maximum end-shear conditions for the span, so that the total impac

ad was not therefore four times that amount. The wind-pressure lowance was also in accordance with the British Standard Specification, which had remained unaltered in that respect. It was suggested at that method was by no means illogical. Bridges, particularly ng structures such as the Lower Zambezi bridge, were generally cated in exposed positions, and even if during a violent storm the nd velocity were such as to derail vehicles the bridge should remain able on its supports. In actual fact the amount of extra material quired to provide the necessary strength and stability under the gulations in force was in most cases of minor account. The ecified method of treatment did ensure that any such material as placed in its most advantageous position. Mr. Handman.

With regard to the economical length of the spans, the varying stribution of the contractors' costs when tendering so affected the st of the different classes of work that if one at least of the other nders had been accepted, the spans would have cost more than the ells and piers, that was to say, the reverse of what actually happened. The reverse would also have been the case with a span of 50 feet, for the ratio of the cost of the superstructure to the cost of e well and pier would then have been in the neighbourhood of ·20 as compared with 0·92 for the 258-foot spans. The actual cost f the latter was £13,368 and the cost of a typical well and pier was 14,555.

The comparison made by Mr. Nichols of the spacing of the track-eepers with the corresponding spacing on Indian bridges was of terest, but the fact that the Lower Zambezi bridge sleepers were cured by stout guard-timbers notched and bolted to each sleeper ight have some bearing on such a comparison. With regard to the nti-creep plates, Mr. Handman had recently visited the bridge d he had been informed that the keys were secure and had needed ttle or no attention since traffic was started on the 1st March, 1935.

In reply to Mr. H. B. Gates, it was extremely doubtful whether e Warren type of truss would have shown any economy over the ratt type for the span-length in question. Intermediate verticals ould have been necessary at alternate panel-points to keep the ngth of the floor-stringers down to economic limits, whilst additional vertical members would also have been necessary at the other nel-points to support the mid-points of the top-chord segments, alternatively, additional material would have been necessary in e top chords to cater for bending stresses due to their own weight. he omission of sub-horizontals supporting the mid-points of the rtical members would have rendered it necessary to have increased e dimensions of those members, which would then have acted as ruts on the full depth of the trusses. The same remark applied ith greater force to the long diagonal struts in the Warren type

Mr. Handman. of truss. The actual trusses adopted were designed, fabricated an erected in such a way as to reduce the secondary stresses to a mini mum when the primary stresses in the members were a maximum and in those circumstances the design presented no particul difficulty. The appropriate outline of Warren truss would not hav been free from redundant members, and methods of design, fabrica tion and erection similar to those which had actually been adopte would have been necessary to minimize secondary stresses.

In conclusion, he wished to point out that the section of the Pape dealing with the geology of the district was to a great extent compile from notes supplied by Mr. F. Dixey, O.B.E., D.Sc., F.G.S., who ha rendered valuable assistance during the preliminary investigation

Mr. Howorth. Mr. HOWORTH, in reply to the Discussion and Correspondenc observed that Mr. Bateson appeared to attach some special signifi cance to the use of the term "contract depth." No special signifi cance was intended. The expression had been in daily use on th works and was understood by everybody to mean 110 feet belo low-water level. No criticism of the engineers' decision to sin certain wells beyond that depth was made or implied in his Pape The bottom in at least two of those wells at 110 feet was definitel too soft to give a satisfactory foundation, and there was never an reasonable alternative to extra sinking. His Paper merely recorde the fact that the wells in question were taken down to extra dept and expressed disappointment that the decision to sink the to extra depth could not be taken at site in time to complete a well-sinking before the 1933–34 flood-season.

He could assure Sir Robert Gales that the easy normal sinkin conditions at the Zambezi were fully realized and appreciated b the contractors' staff. As stated on p. 388,§ the design of th wells would appear to leave little room for improvement when sinkin through sand. The adaptation of all the well-shafts for the applica tion of compressed air naturally relieved the contractors of som anxiety in that it provided an alternative method by which the wel could be got down with certainty to the limiting depths at whic the pneumatic process could be employed with reasonable safet and economy. Unfortunately such difficulties as were met with i connexion with plastic strata occurred at depths greater than th limiting depths for compressed air, and had to be overcome—neve with entirely satisfactory results—by the application of excessiv kentledge and by resorting to excessive pumping. The criticism of the design of the well-curbs was strictly confined to their unsui ability for sinking through stiff though dredgable material withou the application of compressed air. In that connexion there appeare

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§ *Ibid.*

Mr. Howorth.

be some misunderstanding with regard to well No. 24. No aterial was ever met with in that well which was unduly difficult dredge. The difficulty, as with other wells, was to break down ne end-resistance after dredging to depths well below the cutting-dge. He agreed with Professor Inglis that "human ingenuity hould be able to devise some grabbing mechanism whereby the cavation could be carried almost to the limit of the cutting edge," ut he considered that a more hopeful line of approach was offered y the alternative of adapting the design of the working chamber, that existing forms of grabs, combined with more effective wedging ction, could achieve the same object. He was in agreement with r. Risdon that the angle of cone-plate was immaterial in the case of ne small Zambezi wells. They never encountered plastic material and ere never sunk by open dredging to depths sufficient for skin-friction absorb more than a fraction of their available sinking-effort.

Certain information had been asked for with regard to the finishing-vel of the well-heads. The working season proper extended from pril to November, and it might have been possible to arrange the ork so that all founding and plugging of wells in the open river took lace between August and November. In May the river-level night be expected to be about 10 feet, and in August about 5 or 6 eet, above the nominal low-river level specified for the finished vel of the well-heads. For a fortnight or so in November it might r might not fall to nominal low-river level or even below. By a working freeboard" he meant a reasonable height clear above ater. He would suggest that a suitable modification of the ambezi requirements would have been to finish off the well-head roper at 15 feet above low-river level; the last few feet would ave been added after founding and plugging. If necessary, a small orrection-offset would be made in the well itself before the final ages of sinking at some point which would eventually be definitely elow low-river level, but which need not be predetermined except ithin wide limits. As Sir Robert Gales pointed out, deep wells ere not likely to alter in position during the last few feet of sinking, nd he could assure him that on the Zambezi bridge it would have een quite impossible to detect any residual error in the position f the well-heads without the use of instruments. He agreed with im that the somewhat futuristic arrangement of well-heads sug-ested in his comments would be intolerable.

With regard to the question of skin-friction, he could not subscribe the view that any figures given in his Paper supported a case for ne abandonment of Rankine's theory of pressures in granular aterials. Except when cutting-edges were quite clear, skin-friction alues were always obscured by end-resistance to penetration. hat was particularly so in the early stages of sinking, when sinking-

Mr. Howorth. efforts were big compared with skin-friction. The figure of 4·25 cwt suggested on p. 400 § represented a mean value between surface level and 130 feet below ground. Using Rankine's theory th unit value would be zero near the surface, and it followed that th unit value at 130 feet below ground would be of the order of 8 or cwts. per square foot. That value would be obtained with Rankine theory with a material weighing 56 lbs. per cubic foot in wate and having an angle of repose in water of 30 degrees and a coefficier of friction in water between sand and concrete of 0·22. Those value were by no means improbable for the Zambezi sand.

It was perhaps as well to add that with Rankine's theory ther was also available an unlimited choice of other combinations values of angle of repose and angle of friction, which, with the sam weight of material (56 lbs. per cubic foot), would produce the sam value for unit skin-friction. They included and lay between limit of angle of repose 12½ degrees with angle of friction 12½ degree and angle of repose 61 degrees with angle of friction 61 degree That apparent anomaly was due to the fact that in the equation fo skin-friction

$$f = wk.\ \tan\theta\frac{1-\sin\alpha}{1+\sin\alpha},$$

where α denoted the angle of repose of the material and θ denote its angle of friction with the surface of the concrete, the two facto $\tan\theta$ and $\frac{1-\sin\alpha}{1+\sin\alpha}$ tended to neutralize one another.

It would appear that the inference could be drawn that the ang of repose of a material had only a minor effect as regards the ski frictional resistance to well-sinking of that material, and that th important factors were its effective weight, its depth, and th difference between its angle of repose and its angle of friction wit concrete. If its angle of friction with concrete could be varied eith by rapid vibration, as was suggested by Professor Inglis, or otherwis the result was bound to be beneficial. The effect of shot-firing w well known in that connexion.

Although Rankine's theory did not apply without modificatic to non-granular materials, those figures certainly indicated th there were theoretical grounds for suspecting that the same ski friction values might be produced under the same conditions b materials having such widely differing properties as stiff clay ar soft silt. They might also possibly offer an explanation of the wide differing values for skin-friction in various materials which ha from time to time been put forward.

§ *Ibid.*

CORRESPONDENCE
ON PAPERS PUBLISHED IN
FEBRUARY 1937 JOURNAL.

Paper No. 5089.[1]

"The Second-Stage Development of the Lochaber Water-Power Scheme."

By Arthur Holden Naylor, M.Sc., B.Sc. (Eng.), M. Inst. C.E.

Correspondence.

Mr. H. B. Gates, of Perth, Western Australia, referring to the onstruction of the Laggan dam, observed that too much attention ppeared to be given to the effects of cracks in concrete dams. He ad recently visited the Mundaring dam in Western Australia, which ıpplied the well-known pipe-line to Kalgoorlie. The dam was)0 feet high above the bed of the stream, and was about 700 feet in ›tal length (or about 500 feet, neglecting the shallow ends). It ad been constructed in 1900–1902, before so much science had been pplied to concrete structures, without special joints and without a rainage-system. Nevertheless it was safe and in quite good con-ition. The water was about 9 feet below crest-level, and there was o visible seepage. It was a warm day and such small seepage as id occur evaporated too quickly to be visible. The cement skin as just worn off the overfall-section, leaving the stones showing. Mr. Gates.

Some years ago cracks had appeared on the internal face, but ıose cracks had never passed through the dam. It was evident that ›ment-concrete would shrink and might crack when dry, but it velled again when wetted, and the cracks closed. He might also te the case of a smaller dam, for which he had been personally sponsible. That dam was 32 feet high and 240 feet long and had een constructed without special joints. When dry it showed a ack in the centre on the upstream face, but after refilling the spector failed to locate the crack. Another example was that of a ries of reinforced-concrete tanks of monolithic construction. After e first dry-out a few small shrinkage-cracks had appeared, but the igineer had found no sign of a crack when the tanks had been filled. Those tanks had a total length of 282 feet.

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[1] Journal Inst. C.E., vol. 5 (1936–37), p. 3 (February, 1937).

Mr. Gates.

It would, therefore, seem sufficient, even in large dams, to observ
the following precautions :—

(1) Construction-joints should be provided with simple joggl painted, or pasted, with an asphaltic composition.

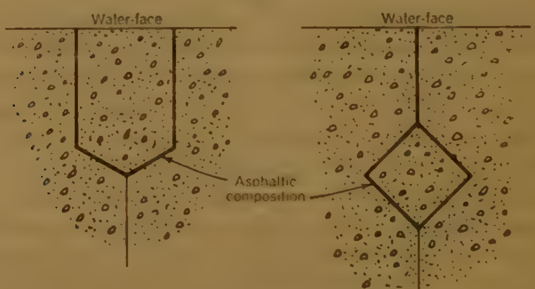
(2) Plenty of plums should be used.

(3) The water/cement ratio should be kept down.

Two alternative designs for self-stanching joints were shown *Figs. 28.*

The value of a gunite lining seemed doubtful. If the main body concrete cracked, the skin was bound to crack also, and it wou seem more profitable to employ the extra money in using pre-ca blocks.

Figs. 28.



Mr. Gullan.

Mr. A. G. Gullan observed that the grey plagioclase granit which outcropped at the Laggan quarry and tunnel-intake, wou have made a very suitable monumental rock, had it not bee marked with dark patches of xenocrysts of the country rock, schist, which had been caught up by the granite in its intrusio The section between expansion-joints 2/3 and 6/7 was a very muc coarser rock, and contained red felspar near each of those expansio joints ; evidence of faulting could be seen, and near expansion-joi 6/7 a fine-textured intrusion (aplite) similar to a dyke was found. O the south side of that intrusion, the rock became very coarse indee The rock in block 3S was very decomposed, as was shown by the lar amount of chlorite present, and a 1½-inch vertical clay seam h been found running east and west at the main fault in block whilst in block 3S ice-markings were clearly seen running nort north-east and south-south-west about 20 feet south of expansio joint 4/5. Beautiful and unusual crystals of calcite had been foun intruded into the granite, which from their appearance had not com in from above, but had been obtained from below. Near the sam

ne a band of schist and an acid felsite had been intruded into the Mr. Gullan. anite at a later phase.

The setting out of the dam had been carried out using as co-rdinates two main base-lines, one 75 feet north of, and the other 75 feet south of, and parallel to, the chord-line. At every 30 feet hainage- and bench-marks had been established.

The physical and geological conditions of the Spean valley would iggest the final position chosen, but he disagreed with the Author n the advisability of sinking trial-pits in the bed of the river. When he ice of the glacial epoch had receded it would naturally take the asiest path and would remove all the loose rock, but at the same me it would leave the top surface greatly marked and shattered, hich was what had been found, and it was not uncommon to find t such a section a fault which meant deep-lying rock relative to round-level. That had been the case at Laggan, and a similar ondition had been found during the construction of the Silent alley reservoir.[1] He felt sure that had accurate rock-levels been vailable before commencing the work a great deal of money and nxiety might have been saved. The position of the cofferdams ould certainly have been selected more favourably, and the heavy xcavation in block 5S, in some places to a depth of 50 feet, might ave been avoided.

By fixing the position of the outfall at a point near the north end f loch Treig, and joining up to the tunnel at adit B, a rock outcrop ould have been used without any fear of scour. That would have ngthened the tunnel, but it would have done away with any soft unnel and costly outfall work, especially when it had been found ecessary to prevent serious erosion, with the possibility of under-utting Treig dam and the railway embankment.

Had the granite pitching at the toe of the dam been found satis-ctory ? The limits of the pitching had been governed by the original lea of fitting spillway-gates in the centre-section of the dam. The ough-dressed pitching, 18 inches deep, was laid in "contoury" courses ith 2 : 1 cement mortar with maximum of 1-inch joints, and the ourses had a minimum width of 9 inches and a minimum length to reak bond of 6 inches. The piece-work rate for laying was 7*s.* 6*d.* er square yard on the horizontal, and 8*s.* 6*d.* per square yard on the ope, the dressing cost having been 2*s.* 6*d.* per cubic foot. It had een decided to build the pitching with the concrete, but that had een found to be impracticable, and a small part had been tried

[1] G. McIldowie, "The Construction of the Silent Valley Reservoir, Belfast ater-Supply." Minutes of Proceedings Inst. C.E., vol. 239 (1934–35, Part I), 465.

Mr. Gullan. out by building up random rubble wedge-shaped walls in 4 : cement-mortar to take the pitching. That method had also bee found to be too difficult, and finally the concrete had been finish off at 2 feet from the surface. To safeguard against the possibili of dislodging the stones, steel dowels $1\frac{1}{4}$ inch in diameter and 3 fe long had been grouted in at 3-foot centres.

The equipment in Laggan quarry had consisted of one 30-inch 1 16-inch slogger and two 25-inch by 12-inch Broadbent crushe having a maximum output of 20 cubic yards of 2-inch aggregate p hour, and an average output of 15 cubic yards per hour. In gene 14-foot vertical holes and 60 per cent. low-freezing gelignite we used, giving an output of 2·4 cubic yards of 2-inch aggregate per l of gelignite. The density of the plagioclase granite was 165 ll per cubic foot. On the dam-excavation about 2 cubic yards we obtained per lb. of explosive.

The average hourly working-output of each crane had been cubic yards of placed concrete over 9 weeks during the busie concreting period. The concrete-mixing plant had consisted two 2-cubic yard "Victoria" Stothert & Pitt mixers, and a 67-cubi foot Henderson bottom-opening skip had been used throughout wi satisfaction. In order to speed up the work, the monotower cran had been fitted with an additional high-speed gear, the hoisting spe having been increased from 110 feet per minute to 178 feet per minut which meant a decrease in load from 7 tons to $3\frac{1}{2}$ tons on high ge at 80 feet radius. Based on the average output, the following we the labour-costs per cubic yard:

Quarry	1*s*. 10*d*.
Ropeway	4*d*.
Mixing	9*d*.
Placing	11*d*.

The Ransome diesel-electric drag-lines were of types 460 and 48 type 480 gave an average output of 20 cubic yards per hour, givi a labour-cost of $5\frac{3}{4}$*d*. per cubic yard. The 145-B.HP. sand-pun had an average output of 7·5 cubic yards per hour, giving a labo cost of 8*d*. per cubic yard. A 110-hour week was worked in t shifts.

Had it been found necessary to keep the channel dredged, especia the extension-end, where it had been almost impossible to maint the slopes? The grey-clay slopes referred to at the top of p. 8 § h an angle of repose of just over $2\frac{1}{2}$ to 1.

The Author, on p. 29 §, gave the water/cement ratios of 7 :

§ Page numbers so marked refer to the Paper. (Journal Inst. C.E., vol. (1936–37) (February, 1937).)—ACTING SEC. INST. C.E.

oncrete and 4 : 1 concrete as 0·55 and 0·4 by weight respectively. Mr. Gullan.
hose figures seemed to be high, and 0·3 and 0·2 respectively would ppear to be more nearly correct.

Mr. J. W. MEARES asked, in connexion with the considerable Mr. Meares. mount of water lost over the spillway of Laggan dam, whether the ossibility of utilizing Lochan na H-Earba had been considered. he plan in the Paper (Fig. 1, Plate 1) was not concerned with that nall loch, and did not show either its marginal contours or its evation. It was, however, so close to loch Laggan that its elevation bove the latter was likely to be small. If that was so, and the anks would give a large waterspread with increased depth, it ppeared that it might be dammed to give a considerable amount f supplementary storage, which could be filled by direct electrical umping from loch Laggan in time of excess and used in times of efect. Whatever the elevation of that reservoir might be, it was ertain that the lift would bear a very small proportion to the head vailable at the power-house. When dealing with such large quanti-es, it was probably impracticable to use any form of automatic ater-lift, such as the ram or the "Hydrautomat," though the latter ad been used in irrigation-canals in India. It became mainly a uestion, not of engineering, but of capital charges and working xpenses, and of whether the extra kilowatt-hours stored in the upper ch were worth while. The principle of main or subsidiary storage umped to elevated reservoirs was already in use in many places.

Mr. T. W. MORAN considered, with regard to the occurrence of Mr. Moran. ermeable cracks due to shrinkage and temperature-stresses, that was possible that a suitable use of gunite on the upstream face of aggan dam might simplify the treatment of the problem, provided at the gunite was carefully designed so as to fulfil its required nctions beyond all possibility of doubt. The conditions to be ealt with in the case of a dam designed and constructed in a similar anner to the Laggan dam were :

(1) *The initial shrinkage due to drying out and cooling.* That shrinkage would be a maximum in the upper part of the dam, where most of it would be accommodated by the radial joints at 45-foot intervals. In the middle and lower levels, however, intermediate cracks were likely to occur between the radial joints, since the tensile strength of the concrete would not be adequate to cope with the distortion of the blocks due to the marked difference in temperature between the face and the hearting.

(2) *Seasonal temperature-movements.* Such movements would be greater in the upper part of the dam than in the lower, and greater at the face than in the hearting. Both

Mr. Moran.

the radial joints and the shrinkage-cracks would g relief to the stresses. The Author and several speak had referred to the possibility of dispensing with facing of 12 inches of rich concrete. That would certai help to reduce the facial cracking. If watertightn were to be the criterion, it would appear that the b place for the rich mix would be as a core in the m thickness of the dam. However, as none of the shrinka cracks appeared to penetrate as much as 11 feet from t face, it would seem, from that and from other data gi by the Author, that the dam would be perfectly wat tight if the rich facing were dispensed with altogether

The functions to be fulfilled by a gunite facing were therefore :

(1) Sealing the radial joints and shrinkage cracks on the stream face, in order to prevent percolation and progressive disintegration due to the dilute acids moorland water.
(2) Sealing the surface pores of the concrete.
(3) Sealing the horizontal construction-joints. The Author not say whether any trouble had been experienced wi percolation through those joints in the case of the Lagg dam, but judging by observations elsewhere, t horizontal joints were more likely to permit leakage th the vertical joints, owing to the difficulty of forming dense mixture at the bottom of each lift.
(4) Providing a protective face to resist wave-action and sl disintegration from frost between top and low wat level.

If the gunite facing were to fulfil those requirements adequat it was clear that it should not fail by reason of (*a*) uncontroll cracking in line with the contraction-joints and shrinkage-cracks the dam, and (*b*) uncontrolled cracking due to shrinkage and te perature-stresses in the gunite itself. It was important to rememb that the critical period was that which elapsed between constructi and filling with water. Once the reservoir filled, temperatu variations would be greatly reduced ; furthermore, the gunite, a the concrete behind, would tend to increase in volume by absorbi water and thus would partially close the contraction-joints a shrinkage-cracks. It would be of interest to know if that had be observed at Laggan dam.

The first step was to select a suitable gunite mixture with a vi to minimizing shrinkage. On the Laggan dam a 3 : 1 (dry) mix

en used, which probably yielded a finished mix of about 2 : 1, d was much richer than the face-concrete. There was a strong *ima facie* case for using a much leaner mix, as near as possible to at of the face concrete; it was quite probable that such a mixture ould be equally as watertight as the 2 : 1 (finished) mix because the rcentage of rebounding sand would be increased, and the particles mposing the finished mix would have been punned home all the ore thoroughly. An increased working-pressure would assist in curing that result. A further point was that the gunite would be eatly improved if some fine chippings were employed, as that ould help to minimize shrinkage. He suggested that the finished oportions to be aimed at should be 3 : 1 or $3\frac{1}{2}$: 1. The dry gauging quired to produce such a mixture would depend *inter alia* upon e fineness-modulus and water-content of the aggregate, and on the orking pressure, but would be somewhere about $4\frac{1}{2}$: 1 or 5 : 1. ie proportion of chippings should be about 20 per cent. of the tal aggregate, and the size from $\frac{3}{8}$ inch to $\frac{1}{8}$ inch, or $\frac{1}{4}$ inch to $\frac{1}{8}$ inch reenings. The admixture of chippings would give a much rougher rface-texture to the gunite than if sand only were used; but that is of no importance when compared with the superior qualities tained. Mr. Moran.

The next consideration was that of shrinkage and temperature-esses in the gunite itself. There was little hope of being able to ovide (economically) enough steel to cover the stresses without owing cracks, and the reinforcement should therefore be designed r the sole purpose of controlling the cracks and distributing them enly. Tensile tests on slabs of reinforced gunite showed that the nite cracked in tension at each of the transverse wires; hence if a ht fabric of, say, 6-inch mesh were used the result would be a ge number of very minute cracks at 6-inch centres. Alternate-bay nstruction might also be adopted with advantage.

Another possible method of minimizing the size of individual acks in the gunite would be to dispose the steel fabric in fairly all sheets, for example about 7 feet square, with a 2-inch gap tween adjacent sheets instead of the usual laps. That would oduce fine cracks at 7-foot intervals, thus relieving the tensile esses, and ensuring that the latter should not become sufficient overcome the adhesion to the concrete. A detail which would quire investigation was whether to break the joints in the perpends in the horizontal edges of the sheets. It would probably be best keep the perpends in line, parallel to the joints in the concrete. e whole proposal was a reversal of existing practice, but it appeared be perfectly logical, and merited consideration.

It was of the utmost importance that the adhesion should always

Mr. Moran.

remain as perfect as possible. To that end, the surface of the concre
should be roughened with shallow hacks close together, or alt
natively the surface might be stripped after applying an aggrega
exposer on the shuttering. Sand-blasting with the cement-gun its
was of very little value for that purpose, as the strippi
effect obtained was negligible. The surface of the concrete should b
thoroughly drenched with a water-jet for some time before t
gunite was applied, and every effort should be made to keep t
gunite continually moistened with a fine spray for about a week aft
application.

A $1\frac{1}{2}$-inch layer should provide adequate watertightness under
head of 180 feet of water, but if the reinforcement were to be situat
in the best position, namely, at mid-thickness, the cover on the wat
face would be slightly less than $\frac{3}{4}$ inch, which was too small f
safety. A 2-inch thickness of gunite would be preferable, as givi
about 1-inch cover to the steel at the centre of the layer.

It appeared that there was not very much danger of cracks occu
ring in the concrete along the horizontal construction-joints, so th
the main object was to secure watertightness across them. Th
reinforcing fabric of the gunite should therefore be so disposed as
lap over the horizontal joint-planes. If individual cases were foun
where cracks had formed, they could be dealt with in the same wa
as described below for vertical joints.

With regard to the vertical joint-planes and shrinkage-crack
the use of gunite could be so arranged as to dispense with the nece
sity for placing the radial contraction-joints in the concrete clos
together than 45 feet. That would be done by postponing t
application of gunite to the latest possible date so as to allow
long as possible for shrinkage-cracks to develop on the face-concret
Joints of a suitable pattern would be provided in the gunite
front of them. He was interested in the type of contraction-joi
described on p. 30 §. He had frequently had occasion to consid
methods of bridging contraction-cracks with a layer of reinforc
gunite, but although the tensile strength of 3 : 1 (dry mix) guni
was at least 500 lbs. per square inch it was difficult to see ho
pronounced cracking could be avoided in a joint of the type de
cribed. He had experimented with buckled copper strips, but it w
an awkward matter to place a joint of that description in a th
layer of gunite. If inaccurately fixed, the gunite tended to crack
line with the edge of the strip, and the joint became usele
A method which he had adopted in lining service-reservoirs w
based on arranging that the gunite would crack in line with t

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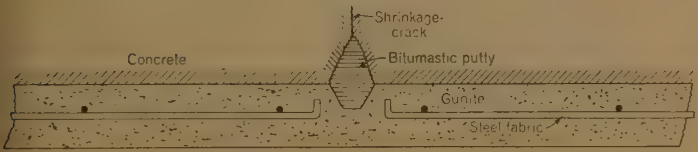
§ *Ibid.*

rack in the concrete. A chase was cut in the concrete $1\frac{1}{2}$ inch wide Mr. Moran. nd 2 inches deep, and filled with bitumastic putty, which was left rojecting about $\frac{3}{4}$ inch proud of the concrete. The gunite reinforcement was cut and hooked opposite the chase (*Fig. 29*). That arrangement controlled the alignment of the crack in the gunite, and the ater-pressure was used to press the putty more firmly home. hat type of joint was made very quickly, it cost little, and it had roved effective. Joints of that type should be sited in line with very contraction-joint and shrinkage-crack. It would be of interest o hear whether a functional use of gunite on the lines indicated bove would have simplified the problems met with in the design nd construction of the Laggan dam.

Fig. 29.



Mr. J. H. WALKER considered that one of the interesting problems Mr. J. H. Walker. n the construction of concrete dams, whether of the gravity type or f the arch form, was that of the prevention of permeable shrinkage-racks. The Author mentioned several methods of mitigating the rouble resulting from shrinkage-cracks, and stated that the problem eased to exist if the dam were built up of pre-cast blocks. The tter opinion was shared by several who took part in the discussion n the Paper. It was worth while considering a further alternative olution, for the full appreciation of which it was desirable to have in ind the following facts :—

(1) Concrete, unrestrained in setting and hardening, would shrink 1 inch or more in a length of 100 feet.

(2) Concrete, subjected to a rise in temperature of 150° F., would expand 1 inch in a length of 100 feet; that shrinkage was far in excess of any probable expansion due to normal rises in temperature.

(3) Shrinkage of concrete was observable within 36 hours of casting, and consequently the shrinkage took place before the concrete had attained sufficient tensile strength to resist shrinkage stresses imposed upon it, perhaps by a solitary large stone displacer, or by a step in the foundation.

Mr. J. H. Walker.

(4) Dock-walls 20 feet wide at the base and made of 8 : concrete, had developed vertical cracks about ⅜ inc wide, spaced at distances apart of from 60 to 80 feet Those cracks were observable before the dock was finish and filled with water.

(5) In dock-walls made with a sufficiently large number stone displacers, as in the case of docks constructed b the Mersey Docks and Harbour Board, vertical crack were not apparent.

(6) A large slab of concrete 8 inches thick, cast on a roug sub-base, would warp up at the edges off the sub-bas Eventually the expansion and contraction, due to varyin weather-conditions, would cause it to crack across th centre and attempt to re-bed itself.

(7) If the above slab had buried in it, below the surface, a laye of rigid circular swaged steel hoops, each laid flat an touching one another, and each hoop 20 inches diamete by 2 inches deep by 18 S.W.G., the slab would not war at the edges. Each hoop, as the concrete encircled b it set and hardened, would become a solid displace consequently the upper part of the slab could not contra in length and breadth; the result would be that th concrete, instead of shrinking bodily to the centre of it mass, would shrink locally and become permeated wit fine, invisible cracks. In other words, the concret would be "crack-controlled."

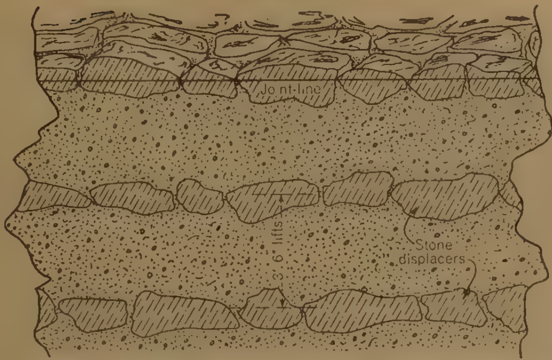
Accepting the above statements as correct, it was easy to appl the displacer-theory to large dams, where the mass-concrete wa brought up in lifts of 3 feet 6 inches.

He would suppose that in the surface of each lift a layer 12 inche deep of large stone displacers, or pre-moulded concrete slabs, wer inserted each stone half in and half out of the wet concrete, an each jammed up against its neighbour (*Fig. 30*). Obviously suc a layer could not shrink in length and width. The concret imprisoned between two such layers, one above and one below was, however, bound to shrink in setting. The inevitable resul would be that the concrete, shrinking while it was weak in tensil strength, would develop minute vertical cracks, or vertical cleavag planes, such as occurred in stratified rocks similarly imprisoned That procedure might or might not be practicable, as firstly a ample supply of displacers would be requisite, and secondly it migh be difficult to locate them in such stratified layers.

If, however, instead of stone displacers, swaged steel hoops wer

Mr. J. H. Walker.

Fig. 30.



PART-SECTION THROUGH CONCRETE WALL CRACK-CONTROLLED BY MEANS OF LAYERS OF STONE DISPLACERS.

used, say 3 feet in diameter by 9 inches deep, made of No. 20 S.W.G. steel, the procedure would be simplified, as such rings could be laid flat and touching one another, half in and half out of the surface of each 3 foot 6 inch-lift (*Fig. 31*).[1]

Fig. 31.

PART-SECTION THROUGH CONCRETE WALL CRACK-CONTROLLED BY MEANS OF LAYERS OF SWAGED STEEL HOOPS.

It would be obvious from the study of *Fig. 31* that by introducing layers of hoops in the mass concrete as shown, the concrete has been forced to form for itself layers of solid concrete displacers, each

[1] Since writing the above, it had been ascertained that the dock-walls of the King's Dock, Swansea, built about 1910, had been brought up in lifts as shown in *Fig. 30*. The plums, however, had been spaced not less than 6 inches apart. So far as could be seen above water-level, there were none of the cracks so observable in dock-walls built of mass-concrete without plums.—J. H. W.

Mr. J. H. Walker.

surrounded by the steel sheeting, which could not be pressed out c shape on account of the weak tensile strength of the concrete at th time it was thus "crack-controlled." Such control resulted in th formation of minute invisible vertical cracks or planes of cleavag (they might also be termed planes of tension), the distance apar of which it was possible to estimate if the relative strengths of th concrete in direct shear and tension were known. For instance assuming that the tensile strength f_t was one-third of the shea strength f_s, then, referring to *Fig. 31*,

$$(AB + CD)f_s = (33 \text{ inches}) \times f_t$$

That was,
$$xf_s = (33 \text{ inches}) \times \frac{f_s}{3}$$

Whence
$$x = 11 \text{ inches}$$

If the total shrinkage of the concrete were 1 inch in 100 feet, th width of an induced crack would be approximately $\frac{11}{1200} = 0{\cdot}009$ inch

Mr. C. A. Hogentogler, Senior Highway Engineer of the Bureau o Public Roads, Washington, U.S.A., referring to concrete road-slabs cast on a rough sub-base, had stated that the shrinkage-cracks initiated by the adherence of the underside of the slab to the ground occurred before the tensile strength of the concrete was 20 lbs. pe square inch. If that were correct, the compressive stress in the stee of the embedded circular hoops did not exceed $16\frac{1}{2}$ tons per squar inch. Considering that the steel could not buckle or otherwise evad the stress, the hoops should function even if that stress were greatly exceeded. Mr. James Williamson in a Paper [1] had mentioned th difficulty of maintaining watertightness at the horizontal joints between successive lifts. It would be seen from *Fig. 31* that the upper parts of the hoops, projecting upwards into the layer above, effectively bonded the two layers together and prevented percolation along the joint.

If it were desired to dissipate the heat of the concrete in each lift the basins formed by the upstanding hoops could be filled with water, which could be left standing for a week or more without delay to the work, provided that the concrete of each lift be pumped or otherwise laid in continuous layers, from one end of the dam to the other. That method was a reversion to the old English practice and differed entirely from the skyscraper method often employed.

Assuming that each steel hoop, weighing $11\frac{3}{4}$ lbs., crack-controlled $1\frac{1}{6}$ cubic yards of concrete, there would then be 10 lbs. of steel to a cubic yard. Estimating hoops in place at £18 13*s*. 4*d*. per ton, or 2*d*. per lb., the additional cost for hoops would be 1*s*. 8*d*. per cubic yard of concrete. Part of the extra cost might be off-set by the us

[1] Second Congress on Large Dams, Washington, 1936.

f stone displacers in the intervening layers of concrete. Such a small addition to the cost of the mass concrete might be warranted, not only in the construction of dams, but also in the many instances of foundations and walls for docks, harbours, etc. where the elimination of unauthorized permeable cracks was desired. Mr. J. H. Walker.

It might be useful to consider the possibility of crack-control of mass-concrete by means of, say, 0·25 per cent. of longitudinal bar-steel and a similar amount of transverse steel. To do that would necessitate the use of $2\frac{1}{2}$ lbs. of steel per cubic foot of concrete, or $67\frac{1}{2}$ lbs. per cubic yard. The cost of that would be 11*s.* 3*d.* per cubic yard, which was far too high to warrant any serious consideration of the use of bar-steel for the purpose of crack-controlling mass-concrete, even if it were successful in doing so, which was doubtful.

Casting the concrete in alternating blocks, and thus pre-forming the joints, or burying thin slab-sheets in it to form induced joints, was only a partial control of shrinkage, and the opening and closing joints thus formed had to be made watertight by copper strips or by other means.

An interesting feature of the Laggan dam was the facing of the 7 : 1 concrete with an outer layer of 4 : 1 concrete, the latter being made with a small-sized aggregate, consisting of stones 1 inch and downwards. As a further precaution against percolation of water, the 4 : 1 concrete facing was itself faced with a still more richly mixed reinforced-concrete coating of gunite, using aggregate of $\frac{1}{4}$ inch and downwards. Such elaborate precautions suggested that the method of facing the dam could be much simplified, if only for the reason that the correct solution of any problem was usually very simple. The experience gained in the construction of concrete roads might help, as their surfaces were not only subjected to the wear and tear of traffic, but also to extremes of weather-conditions.

Concrete roadways had been made in Edinburgh in 1873, for one of which, $\frac{1}{4}$ mile long, the maintenance costs over a period of more than 60 years had been less than £50. Those road-crusts were 6 inches thick and were without reinforcement, the bottom 3 inches being ballast-concrete with the top 3 inches macadam concrete, the aggregate of the latter being largely of sizes from 2 inches to 4 inches, which to-day showed as a large-sized mosaic. Those examples of excellent concrete roadways had been forgotten, and when some 40 years later interest in concrete-road construction had been revived, the roads were formed of large rigid slabs up to 15 inches thick, laid on brown paper, etc., and reinforced with 21 lbs. of steel per square yard, such slabs having coarse aggregate in the base and finer aggregate in the surface concrete. One of the faults of such slabs was that they warped upwards at the edges, and gave bad running

Mr. J. H. Walker.

over the joints, so that sooner or later, they had to be tar-carpeted. About 15 years ago the Edinburgh example had been unknowingly followed, as the reinforcement had been omitted as costly and unnecessary, with the result that there were many miles to-day of such roads in excellent condition, which being devoid of warp, gave most satisfactory running to motor vehicles. The only departures from the Edinburgh system were that the lower part of the slab was from 4 to 6 inches thick, and contained the usual aggregate of $1\frac{1}{2}$ inch and downwards, whilst the surface coat was mainly of $2\frac{1}{2}$-inch broken granite.

The success of such a simple form of construction might be sought in the facts that concretes rich in cement, and concretes made with small aggregate, shrunk more than concretes weak in cement and made of large aggregate. Such surfacings of large-sized macadam-concrete might be readily conceived as a conglomeration of small plums or displacers, wherein each large stone was too deeply embedded to be hammered or sucked loose by iron- or pneumatic-tired vehicles ; due, however, to such comparatively large aggregate, the concrete in shrinking not only yielded minutely around each stone, instead of developing large cracks widely spaced (that was to say, it was self-crack-controlled) but the surface concrete lacked strength to warp the edges of the slab off the road-base, with which it had to remain tightly in contact. The slab was freed from the beam and cantilever stresses inseparable from the warped and rigid, and therefore imperfectly bedded, reinforced slab.

A further feature, worthy of notice, was that the macadam-concrete surface, unlike small-aggregate concrete, gave perfect adherence to a brushed coat of dehydrated tar or bitumen, which was waterproof and to a certain extent, plastic. That experience might be applied to the facing of the Laggan dam, where the 7 : 1 concrete with 2-inch aggregate was faced with a richer mix of 4 : 1 concrete containing smaller aggregate, and the latter was faced with a still richer mix with still smaller aggregate ; the principle of the successful concrete road might be used and the 7 : 1 concrete might be faced with a 6 : 1, or even a 7 : 1, mix of concrete, containing granite aggregate varying from 3 inches to $1\frac{1}{2}$ inches, which could be brushed over with a coat of dehydrated tar. The latter should, however, not be necessary, neither might it be æsthetically desirable. In support of that suggestion, it might be of interest to note that the Port of London Authority, previous to 1920, had faced new dock walls of 8 : 1 mass-concrete with a 6-inch thickness of 4 : 1 concrete containing small aggregate. Since then, they had increased the size of the aggregate in the facework to that of $2\frac{1}{2}$-inch macadam, which was the same size as in the surfacing of all their concrete

Mr. J. H. Walker.

oads. The result was good, and moreover the rubbing of barges longside the walls did less damage.

Mr. Ronald Walker.

Mr. RONALD WALKER would like to make reference to the following vording at the top of p. 29 §: "The grading of the aggregates was nalysed and compared with Fuller's maximum-density grading, ut no attempt was made to separate out and proportion each rain-size of the material. . . ." He thought that it would be of nterest to have a diagram showing: (*a*) what variation was btained in the sieve-analyses of the aggregates, and (*b*) to what xtent the ascertained limits of the sieve-analyses deviated from 'uller's maximum-density curve.

With regard to variations in sieve-analyses, the experiments arried out by Professor H. N. Walsh [1] showed that comparatively mall variations called for increased cement to obtain the desired vorkability and strength. Assuming that the sieve-analyses f the aggregates had varied materially, he would further uggest that it would be of interest to have, in the form of a second liagram which could be compared with the first, information as to ow densities and strengths had been affected by variations in ieve-analysis.

He would welcome the Author's observations on the following uggestion: with regard to the construction of large concrete tructures for the retention of water, requirements would best be net by the use of aggregates conforming to such gradation-urves as would, in conjunction with the minimum contents of ement (preferably coarsely ground), result in workable concretes f maximum density, with minimum liability to be adversely ffected by temperature-effects.

Applied in practice, the foregoing would mean, so far as the radation of aggregates was concerned, that the ideal gradation-urves would be found by experiments, and plant (which was now vailable) would be employed to ensure that all aggregates used vould constantly and reliably conform thereto.

Mr. Williams.

Mr. G. BRANSBY WILLIAMS observed that British dams, until ecently, had been chiefly notable for the cautious (he might almost ay timid) lines on which they had been designed, and for their roportionately high cost. It had been claimed as a merit in Papers ead before The Institution that the maximum stress-intensities in articular dams had been restricted to figures much below those vhich the material of which they were composed could safely with-tand, and much time and labour had been expended in elaborating

§ *Ibid.*

1 Bulletins of Inst. C.E. Ireland, May, 1933, and April, 1936.

Mr. Williams. profiles which had been intended to ensure those uneconomic stresses and had been expected to obviate the possibility of tensile stresses in parts of the dams where tension could not occur. It had at length come to be generally realized that a plain triangular section was suitable for all gravity dams of sizes built, or likely to be built, in Great Britain. The Laggan dam was, practically speaking, triangular, and its profile was safe and economical.

The Author mentioned that the alternative of an arch dam had been considered. He presumably meant a composite arch-and-gravity section dam; a single-arch dam would not have been suitable. He stated that, in the case of the Laggan dam, 700 feet had been found to be the limit beyond which there was no advantage in a dam of that type, and that the wisdom of longer arched dams was open to question in view of the lack of knowledge of the stresses set up by temperature-changes and shrinkage. That view did not seem to be shared by engineers in the United States, who had projected considerably longer dams of the arch-and-gravity type and who had apparently found those more economical than gravity dams. It was not easy to understand why a lack of knowledge of temperature and shrinkage-stresses should be more prejudicial in the case of dams consisting of arched centres and gravity approach-lengths, than in dams of the ordinary gravity type. It might be noted that the first suggestion for the construction of combined arch-and-gravity dams in Great Britain, seemed to have been made by Colonel J. Pennycuick, the builder of the Periyar dam, at a discussion on a Paper read before The Institution.[1] Until recently that suggestion had not been followed up, but the Galloway dams had shown its value.

It would be of interest to know whether the alternative of a multiple-arch dam had been considered for the Laggan dam. Such a dam would have been much cheaper, and objections to it, with the improved quality of cement concrete available to-day, were largely based on prejudice. A few years ago an Italian engineer, as a result of his experience of the multiple-arch dams that had been built in Italy, had stigmatized the masonry gravity dam as an "economic crime." His statement might perhaps be regarded as too sweeping but it was likely that those solid masses of material would be superseded in the future by lighter, more economical, and more scientific forms of construction. There was little doubt that in many places where masonry dams up to 200 feet high had been built, or proposed

[1] L. A. B. Wade, "Concrete and Masonry Dam-construction in New South Wales." Minutes of Proceedings Inst. C.E., vol. clxxviii (1908–1909, Part IV), p. 1.

Mr. Williams.

ultiple-arch dams would have been quite satisfactory and much heaper than gravity dams.

The Author had suggested the possibility of having an all-siphon pillway. The objection to a spillway of that type was its lack of lasticity. The Committee of The Institution on Floods in Relation o Reservoir Practice had recommended that the design of spillways hould be based on the discharge during a probable maximum flood, nd that what were termed " catastrophic " floods should be provided or by a margin above the anticipated flood-water level in the eservoir, within which the water could rise if necessary.[1] That eemed to be the common-sense method of dealing with the problem, nd in the case of an ordinary spillway the rise necessary to discharge wice the probable maximum flood-flow would usually not be xcessive. If, however, an all-siphon spillway were in use which ad been designed on the same basis, a catastrophic flood would ompletely overwhelm it and a major disaster might result.

It could not be said that the estimated flood-discharges in the ochaber scheme erred on the safe side. It seemed rather misleading o state the run-offs in terms of inches per day, for the periods of oncentration were generally less than 24 hours, and on the Treig atchment they were very much less. According to figures Mr. Villiams had adopted for that type of catchment, the probable aximum flood-discharge from the Treig catchment would be about inch per hour, and from the Laggan catchment $\frac{1}{6}$ inch per hour. n a " catastrophic " flood those figures might be considerably igher. At all events it would hardly be denied that such rates of ischarge were physically possible, and in a work of the type under onsideration any flood even remotely possible was a contingency o be reckoned with.

The Treig dam was a combination of a rock-fill and an earth-fill am, somewhat similar to some dams constructed in America, and as of a type which, so far as he knew, had not previously been onstructed in Great Britain. It would be interesting to know on hat basis the thickness of the reinforced-concrete corewall had een determined. In most dams where such corewalls had been sed, their thickness had been decided by what had been found tisfactory in other similar dams. Efforts had been made to arrive the correct thickness by calculation, but they had not been very ccessful, and had obviously started from faulty premises.

The information regarding the temperature-rises was interesting, d would be valuable for future reference in connexion with other ms. The length of time required for the dispersal of the heat

[1] Interim Report, Inst. C.E., 1933.

Mr. Williams. produced by the setting of the cement in a comparatively small da illustrated the necessity for the artificial cooling-devices which ha been adopted for a dam of the size of the Boulder dam.

The agreement between the calculated and measured deflexio was as close as could be expected, when allowance was made f the uncertainty regarding some of the assumptions on which t former were based. The Author suggested that the method ascertaining deflexion described in the Paper was justified by t assurance of safety obtained from it; it hardly seemed that an such assurance was required in the case of a dam designed an constructed by the methods employed for the Laggan dam. T failure of that dam during the next few hundred years seem unlikely, unless caused by some cataclysm of nature. If such we to occur, the information obtained from measuring the deflexio under normal water-loads would not be of much help in averting t disaster.

The Author. The AUTHOR, in reply, observed that since the construction of t works no maintenance had been necessary in the dredged chann from loch Laggan.

Mr. Meares had suggested the damming of Lochan na H-Earb to give supplementary storage and thus to reduce flood-loss. T difference of level was over 300 feet and it would be quite uneconom to pump to such a supplementary reservoir. It would, however, possible to regulate the gravity flow therefrom so as to render t extra storage effective. The small amount of additional pow would probably be more than counterbalanced by the capit expenditure involved.

Mr. Williams' remarks appeared to imply that insufficient pr vision had been made for flood-discharge. In the Discussion M Gourley had dealt with that point at some length, and the Author reply thereto (p. 81§) covered the present criticisms. It was interes ing to contrast Mr. Williams' implication that flood-conditions Treig dam were particularly underestimated with Mr. Gourley opinion that, while there might be some doubt in the case Laggan dam, there was an ample margin at Treig dam.

The Author was in full agreement concerning the danger attendi the use of an all-siphon spillway of insufficient capacity, and he ha elsewhere emphasized that very point. Such a spillway should, his opinion, have sufficient capacity to provide a considerable marg beyond a "catastrophic" flood, but that surely did not necessari mean that an all-siphon spillway should be ruled out of consideratio

In the preliminary designs both single-arch and composite arc

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§ *Ibid.*

ıd-gravity dams had been considered. When the sides of the The Author. ılley were steep the advantage of the latter type was largely offset ŕ the longitudinal yield of the gravity-section abutments. The ıthor agreed that arch dams of greater length and less than gravity ction might be constructed without much danger of actual failure, ovided that cracking and high stresses could be regarded with uanimity. He did not suggest that a lack of knowledge of temrature and shrinkage stresses was more prejudicial in the case of ched dams, but rather that in view of that uncertainty it might unwise to adopt a section less than was given by a gravity design. multiple-arch type was not suitable for an overflow dam. The uthor did not agree that such a type would have been much ıeaper. It was doubtful whether, in such a V-shaped valley where e section of maximum depth was short, either the multiple-arch the Ambursen type would show any economy.

Mr. Gullan considered that a trial pit should have been put down the bed of the river at Laggan dam. The rock-levels were not ery different from those anticipated. Even had such a pit shown e lowest point of the rock-surface to be at a considerably greater pth the location of the dam would not have been affected, and e difficulty of sinking such a pit would have been commensurate ith any saving that might have been effected in connexion with e cofferdams. The Author did not understand the reference to e heavy excavation in block 5S, since that had been dictated lely by the level of the sound rock.

The Author agreed with Mr. Ronald Walker that information lating variation of strength to variation of sieve-analysis would of interest. Since strength depended more upon water/cement tio than upon grading, very accurate control of the former was sential before conclusions could be drawn concerning the effect grading. That had not been practicable owing to variation in oisture-content of the aggregates and the difficulty of accurately uging the added water. Attention had been directed to the proximate maintenance of a grading known to be satisfactory, and had not been feasible to conduct a research into the effect of partures from that grading. The high densities consistently aintained were evidence of the successful results obtained. The ggestion for optimum grading appeared to the Author to be mewhat platitudinous in form. The question of coarsely- versus ıely-ground cement was still open, the slower evolution of heat th coarsely-ground cement being offset by improved workability th finely-ground cement. The suggestion omitted all mention of ıter/cement ratio, upon which the strength so largely depended, d it would allow a concrete in which the cement was replaced by

The Author. finely-ground sand. The water/cement ratios given were approxima in that an estimate had been made of the water-content of t coarse aggregate. The figures suggested by Mr. Gullan, name 0·3 and 0·2, were impossibly low even for concrete vibrated in laboratory.

Several different views had been expressed on the question cracking in dams. Mr. Gates thought the importance of the matt to be overrated and special construction to be unnecessary. T value of gunite he considered doubtful, but Mr. Moran favoured i use. Mr. Walker favoured a dam permeated with cracks. Th cracking was not experienced to any great extent in many old da was explained by their slow speed of construction, which allow adequate dissipation of internal heat. Such a policy was not n economically practicable. There was abundant evidence th cracking might cause unsightly seepage, and with moorland wate progressive deterioration of the concrete might ensue. The Auth understood that no leakage had been observed on the downstrea face of Laggan dam. He did not feel competent to comment on t interesting suggestions put forward by Mr. Moran for possib improvement of the gunite mix and mode of application. It was interest to note that gunite cracked, as would be expected, opposi the cross-reinforcement. That was a satisfactory result as far protection of cracks in the dam was concerned, as it was extreme unlikely that those hair-cracks would coincide with the cracks the face of the dam. Except where separated from the concre of the dam by a slipping layer, the gunite was bound to contra and expand with the concrete and the percentage of reinforceme was of little moment, so that there would appear to be no advanta in forming it in 7-foot squares as suggested. At contraction-join where sealing had been effected by a joggled copper strip t tendency for cracking of the gunite opposite the edges of the str was minimized by a local thickening of the gunite cover.

Mr. J. H. Walker advocated the avoidance of obvious cracki and the minimization of contraction by ensuring the formation innumerable hair-cracks. His suggestion of steel hoops h embedded in the surface of a lift apparently killed three birds wi one stone—controlling cracking, ensuring proper curing and formi an excellent bond with the next lift. As applied to roads a situations where structural strength was not of great importan there was no doubt much to be said for his principles. Applied a dam, the Author considered it would be a mistaken policy. On a hair-crack had formed there was a tendency for it to devel owing to the concentration of stress at its growing tip. Mr. Walke construction would tend to cause a dam to crystallize out into

lumnar structure like basalt, and the strength of the dam would en be gravely impaired. The use of layers of stone displacers most in contact was not so objectionable from that point of view, it there were practical difficulties. With the low water/cement tio desirable for dam-work it was not possible to bed displacers tisfactorily without allowing a fair space between adjacent stones id without a considerable expenditure of time and energy. The Author.

It was difficult to see how the use of a lean upstream facing with rge aggregate could prevent the formation of definite vertical acks due to the thermal expansion of the heart of a block. Such facing might contract less if allowed to dry out, but contraction ould in any case be prevented by keeping the face damp until e dam was filled. A crack-controlled facing as suggested would, the Author's opinion, be more vulnerable to attack by aggressive oorland water notwithstanding a coating of de-hydrated tar.

The granite pitching at Laggan dam extended over that part of e curved toe where the overflowing water had maximum velocity id turbulence. It was too soon by several centuries, he hoped, to sess its behaviour. It would be more correct to describe the rlier methods of constructing the pitching attempted as being und inconvenient rather than impracticable. The contractors had en allowed to lay the pitching afterwards, provided that adequate nd was ensured by the use of steel dowels. There appeared to be objection to that method since, as there would be no contraction ie to setting- and drying-shrinkage in the layer of pitching, it ould bear its due share of the stresses set up in the dam when full.

The alternative location of the Laggan–Treig tunnel outfall in ch Treig itself had been carefully considered. In the light of the formation then available from borings that was the more expensive heme.

Treig dam was functionally a rock-fill dam. There was little idance as to the thickness of concrete corewalls. It had to be pervious and capable of resisting bending and shear stresses with e water-pressure acting down to the rock foundation. In the case Treig dam, with two-thirds of the corewall below ground, the ickness had been determined from considerations of trench-dth, a minimum width of 6 feet having been maintained at the ck-surface in order to ensure the formation of a satisfactory key th the sound rock and room for dealing with influx of water during nstruction.

Paper No. 5076.[1]

"Pre-Stressing Bridge Girders."

By Herbert John Nichols, B.Sc., M. Inst. C.E.

Correspondence.

Mr. Everall. Mr. W. T. Everall observed that mention was made (p. 103 that the camber had been found to be reduced permanently by 0· inch, and that the reason for that was not altogether clear. It w found that the deformation-stresses showed little change in individ members, but that there was a substantial permanent set in members, except the top chord. Was that permanent set account for by the slip in the rivets at the joints and connexions due to t loads applied immediately after and during the erection ?

Considerable initial axial stresses were imposed by the p stressing, and slip in the rivets would occur in the reverse directi to the extent of that loading. He considered that the movemen due to that slip were in addition to those caused by the applicati of the normal load, and that that would account for a large proporti of the 0·42-inch reduction in camber. The importance of refineme in the fabrication of the girders was stressed by the Author, and t details of the tolerances permitted, given in Table III (p. 115 were of considerable interest. The degree of accuracy and t limited tolerances under which the Nerbudda bridge girders had be manufactured were essential features, and had been largely respo ible for the excellent results obtained from the pre-stressing meth of erection employed on the bridge. Bridge-engineers in In and elsewhere owed much to the firm of Messrs. Braithwaite & (Engineers), Ltd., and especially to the engineer, Mr. T. Douglas, the way in which they had developed such a high standard construction.

As the system adopted for obtaining girder-work suitable for p stressed conditions of erection might be of interest, he append

[1] Journal Inst. C.E., vol. 5 (1936–37), p. 91 (February, 1937).

§ Page numbers so marked refer to the Paper.—Acting Sec. Inst. C.E.

e following notes on the jig-method of fabrication, as developed Mr. Everall.
 Mr. Douglas and used in India.

Notes on Jig-Methods of Fabrication.

(1) To obtain the best results, both as regards workmanship, cost of production, and output, experience in England and India has proved that the system must be made as foolproof as possible. This applies not only to the actual making of jigs, but also to their application during all stages of the job.

(2) Perfect workmanship is demanded by the development of pre-stressed bridge-work.

(3) Jigs are to be made of steel, and fitted with case-hardened bushes.

(4) Some notes on the making of jigs :—

(*a*) All setting-out is done by tape-measurement and by trammels, on the bench of a totally-enclosed jig-shop, in an even temperature.

No setting-out is done on the template-shop floor.

(*b*) One tape only is used (having an N.P.L. test-certificate). It is fitted with a spring-balance tension handle, and all measurements are made with a pull of 10 lbs.

(*c*) Standard drilling strips (bushed) are made in the fitting shop for all running pitches. These strips are applied to a minimum number of pilot-holes, previously marked off and drilled by sensitive drills, and checked by trammels and dividers.

(*d*) Jigs are first made for all the small fittings, and these in turn are later applied to larger main-member jigs.

(*e*) The maximum wear allowed in bushes is 0·008 inch, and jigs are tested daily with a " go and not go " gauge.

(*f*) Groups of holes in web-members (connecting to main gussets) are located on a specially-designed surface-table, and drilled after the members have been completely shop-riveted. It is almost impossible to guarantee exact length-measurement and absolute squareness of holes in all sides by the use of tapes and by " squaring over " from one surface to another.

(*g*) When chord-members have been fabricated in halves, these halves are assembled with the use of special cast diaphragms, machined all over, and drilled for insertion of turned bolts through the machined diaphragm-castings.

(*h*) The maximum variation in any length-measurement and/or test of squareness after complete fabrication is plus or minus $\frac{1}{64}$ inch.

(*i*) In ordinary bridge-work, strictly interchangeable joints can be made with turned bolts and parallel drifts with a maximum opening, measured over four corners, of 0·005 inch.

A special taper gauge and straight-edge is used at the milling machine, in conjunction with temporarily-inserted machined diaphragms, to prove the accuracy of milling.

(*j*) In addition to the guarantee of squareness, length, etc., temporary machined diaphragms used for assembling

Mr. Everall.

ensure uniformity of width between webs of such built-u members.

(5) Jig work has proved successful in England as regards quality of wo manship, and for economic reasons; in consequence (and in country where skilled-labour wages are high) much of the workl reduced to semi-skilled or unskilled labour standards.

In India, labour is cheap, and skilled labour is not so good or plentiful as in England, but by the adoption of a foolproof jig-syste the Indian workman can produce finished bridge-work equal quality to that of any other country.

(6) In the case of big repetition bridge-work such as Nerbudda, Jumn Bally, etc., the process of marking-off and drilling to centre-po was eliminated entirely.

(7) For the class of repetition-work referred to, the manufacturing co are lower than for ordinary plated bridge-work, and the relati increase in output is not less than 25 per cent.

(8) Other advantages of strictly interchangeable work are :—

(*a*) A system of marking, resulting in all identical pieces beari one mark only regardless of the span to which they m belong. This permits easy reference throughout off routine-work, and permits speedy handling of compone parts during fabrication.

(*b*) Interchangeable work requires very much less stacking spa both in the works and at site prior to erection.

Handling and sorting charges are thus reduced to absolute minimum.

(*c*) Only in the case of "plated and match-marked" work is necessary to paint each span a different colour. Int changeable work renders this unnecessary, as well as t very costly and laborious process of "match-marking."

(*d*) Erection at site can be performed by reference to one or t fully-marked small-scale plans, and without any referen to numerous detail-drawings.

(*e*) Strictly interchangeable work produced by jig-methods giv the same degree of accuracy throughout, whereas, in plat (marked-off) work, the shop-riveted portions are inferi in workmanship to site-riveted connexions which ha been reamed in position.

(9) The design of jigs and their control, when being made, is usua entrusted to one man, and by means of special cross-checking completed jigs, their accuracy can be assured before fabricati commences.

(10) Modern jig-methods have made "shop-erection" unnecessary, a in recent cases where temporary erection of one span has be required, this has often been done during the closing stages of t contract; namely when from 50 to 75 per cent. of the whole tonna has been delivered to the site.

(11) When jigs for repetition work have been prepared and check perfect workmanship results from their distribution to and fab cation in any number of works; for example, Jumna bridge w fabricated in random parts (not in spans) at two works 1,100 mi apart.

Mr. Everall.

This system has the very important advantage of halving the normal delivery of one works, as in the case of Jumna, simply because 50 per cent. of the steelwork was delivered from Bombay, and 50 per cent. from Calcutta.

(14) Some points to be observed during fabrication :—

(*a*) *Clearances* are to be made as an allowance for the gathering of thicknesses, for the insertion of web-members between main gussets, and also end clearances in such members as cross-girders connecting into chord-members.

(*b*) *Riveting.* A careful selection of riveting machines exerting pressures of 25 tons, 35 tons or 50 tons, according to the type of work being riveted, is very necessary.

(*c*) *Service Bolts.* Use a liberal supply of service bolts and high-tensile parallel drifts during assembly of all work in the shops or at the site.

(*d*) *Cooling-out after riveting.* To avoid distortion or permanent set in large members, pay special attention to the correct laying-out of members on level stallages, after shop-riveting.

(*e*) *Distortion* (*during temporary erection*). In pre-stressed bridge-work, care must be taken if drifting is done ; in the case of end rakers where the stress induced is fairly considerable, it is advisable to do this by means of union-screws and not by drifting.

(*f*) *Span erection.* Wherever practicable it is advisable to erect in the following sequence :—

(i) Lay out the bottom chord, complete with the floor-system and all lower lateral bracing, on grillages which have been set dead level, completing all the chord-joints as the chord-erection proceeds.

(ii) Insert all the web-members.

(iii) Lower the camber-jacks to give the correct theoretical camber-ordinates.

(iv) Working alternately from the centre-line of the span towards each bearing end, erect the top chords together with the top laterals and/or the sway-bracing.

(v) Complete the top-chord joints as and when the adjacent pieces are placed together, finally closing the span (end rakers) at the bearing ends of the span.

(*g*) *High-tensile steel.* Although its workability is almost equal to that of mild steel, it is advisable to drill high-tensile steel at a lower speed than that used for mild steel.

(*h*) *Drills.* It is important to have drill-grinding done by a mechanic on an automatic drill-grinder ; this results in accurate drilling, less wear on steel bushes, and gives increased life to the drills.

(*j*) *Rivets.* They must be heated uniformly to the correct temperature, being careful to remove all scale before driving the rivet. Obtain uniformity in the dimensions of rivet-snaps.

Mr. Everall.

When driving close-pitch rivets in long members, driv "stitch" rivets first at 2-foot intervals, and then retur to drive the remaining rivets.

The Author had made some valuable suggestions for modifying th design of floor-systems, with a view to eliminating cross- and stringer girders and timber sleepers, by adopting a transverse-trough floor That, however, would mean that the metal required for the stringer girders would be transferred to the chords. Mr. Everall agreed tha under that conversion the concentration of metal in the chords would increase the lateral stiffness of the bridge.

In attaching the running rails to the transverse troughing, h presumed that the rails would be welded in lengths equal to tw spans, and that expansion and contraction would be allowed for b means of slip-joints situated at each end of the welded length of rail

The Author might have some views on that matter, and it would also be of interest to know the saving in weight of metal, etc., which he hoped to obtain in adopting that type of bridge.

Mr. La Touche. Mr. J. N. D. LA TOUCHE noted that the bases which had been use for the strain-gauges employed in the experiments had been 20 inche and 10 inches. He suggested that a base measuring $E/1,000$ in inches would be preferable, as an extension or contraction of 0·00 inch then meant a stress of 1 ton per square inch. The stress in fact could be read direct without calculation. He had adopted tha length for his own strain-gauge, taking the value of E at 13,000 ton per square inch, and the accuracy of the results had been proved in more than one testing-machine.

The effect of change of temperature on such measurements wa of considerable importance. When applying tests to plate-girder in India with his strain-gauge, an attempt to obtain the maximum effect of a day's traffic had been made by leaving the gauge on th girder all day. His idea had been that the parts of the gauge would expand equally with the part under test, but it had been found tha the girder expanded more than the gauge. The reason for that was he thought, that the gauge had had a draught of air all around it and had not absorbed as much heat as the girder; the fact tha the gauge had been of bright steel, whilst the girder had been black might also have had some effect. That difficulty might be avoided by taking observations at night.

He was glad to see that direct measurement had been added t calculation as a check on the security of the bridge; he had alway advocated such a step.

Mr. Toms. Mr. A. H. TOMS observed that the method of pre-stressing whic had been advocated by the Consulting Engineers was theoreticall

ound, but as applied it had suffered from certain practical limitations which the Author had described. It would appear that in order o increase its effectiveness the following modifications were necessary :— Mr. Toms.

(*a*) Chord-joints should be so arranged that they had adequate rigidity at the time when the chords were being brought to the desired camber.

(*b*) Web-members should be brought into correct alignment at the joints by straining-devices rather than by drifting.

(*c*) The accuracy of alignment should be checked by accurately-scribed marks on members and gussets.

(*d*) Sub-members should be eliminated as far as possible, and the K-type truss adopted in cases in which the full panel-length of the unsubdivided N-girder would be uneconomically long for the floor-system.

The Author did not describe the procedure adopted for drifting members into position at the joints, but as many drifts as possible should be used at each joint consistent with adequate bolting, and in order to minimize distortion of holes they should be driven in strict rotation so that the shear due to moment on them would be distributed as evenly as possible. The procedure in removing drifts and riveting-up was equally important, the centre holes in a joint being filled first and the drifts in the outer holes retained as long as possible.

On p. 114 § the Author stated that an error of 0·010 inch in the length of a member 40 feet long would result in an undesired axial stress of 0·28 ton per square inch. That would be true if the member had to be forced into place in a perfectly rigid frame, but in normal cases the frame was flexible until the member was in place, and consequently the effect of such an error was negligible.

With regard to the use of Whittemore strain-gauges (mentioned on p. 99 §) it would be of interest if the Author could give further particulars of the instruments and the manner in which they were used, and also if he would state whether any trouble had been experienced from hand-effects.

The Author's proposals for using web-members with lacing in the plane of the trusses appeared to be unsound for the following reasons :—

(*a*) In general it would be practically as difficult to drift up the end connexions of members as in the normal case, since the components of the member had not only to be bent

§ *Ibid.*

Mr. Toms.

but had also, in most cases, to be axially stressed by the drifting. For instance, in the case of a member bent in single curvature with equal and opposite moments at the two ends, the shear was everywhere zero, and consequently the lacing-bars were inoperative. The end rivet-groups would in that case still have to do all the work of pre-stressing exactly as in the original method. In the case of a member bent in double curvature it would certainly be easier to make the end connexions with the lacing loose, but when that had been done it would be found that the amounts of the strains by which the lacing-bars had to be drifted would be so small that they would be inadequate for the production of strain in the main section. The member would therefore be riveted up crooked but practically unstressed.

(*b*) In view of the foregoing, in addition to fulfilling their normal functions, lacing systems in the plane of the truss would have to be capable of withstanding the shear forces corresponding to the deformation-moments.

(*c*) Connexions would in general be less satisfactory from the point of view of design.

With regard to the device used for mangifying the axial deformations of the model-members, the Author stated (p. 134 §) that since the two components were rigidly attached at points *A* and *B* (*Fig. 27*, p. 133 §), a moment tending to bend the member would cause no movement in DE. That was only true provided that no relative rotation of the members DE could occur about the points A and B. If the only connexion at those points was by a single screw, then, unless held by friction, each of the two components would deform in the manner shown in *Fig. 30*, with consequent loss of rigidity in bending of the member as a whole.

On p. 108 § the Author stated that on the model itself it had been a simple matter to measure the deflexion-angles. It would be of interest if the Author would describe how that had been done, since the accurate estimation of the directions of the tangents to the members at joints would appear to be very difficult without the use of special tangent-indicators permanently fixed to the joints.

With regard to the Author's proposals for the reduction of floor-interaction stresses by the substitution of trough floor-plates supported by the chords in place of the normal floor-system, it should

§ *Ibid.*

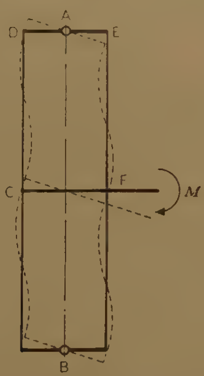
be observed that, unless specially supported, that type of flooring would cause secondary stresses in all members due to :— Mr. Toms.

(*a*) bending of the supporting chords as beams between the panel-points, and

(*b*) torsion of the chords and transverse bending of the web-members due to the deflexion of the troughing and eccentric loading of the chords.

The Author's criticism of the method of calculating the stresses and strains in trusses on the principles of rigid-body statics, and his statement that the actual stresses in sub-members were reduced by the loads carried in bending by the chords to which they were

Fig. 30.



attached, were correct in the case of a truss erected in the ordinary way and not pre-stressed. However, in the case of a truss which was initially pre-stressed, the chords were deliberately bent to conform with the cambered profile, there being local deformations to suit the cambered lengths of the sub-members. If the pre-stressing were correctly carried out, then under the full load for which it was pre-stressed the chords of the truss would be perfectly straight. Consequently, under that load, no load would be taken by the chords in bending, and all members would be subjected to the full forces as calculated by rigid-body statics.

With reference to the economy of the use of steel-bushed jigs, experience in the erection of spans fabricated in that manner (and in which all similar parts were therefore interchangeable) had shown that, in bridges in which there were several similar spans or many similar parts, the time saved in fabrication and erection offset any extra cost of jigs. The question of the cost of jigs as an

Mr. Toms. objection to pre-stressing was therefore ruled out in the case of such bridges.

The Author. The AUTHOR, in reply, observed that it would be agreed that the notes on jig-methods of fabrication by Mr. Everall constituted a most valuable contribution to the subject under discussion. It was probably the first time that such information had been made available, and he would like to see it used as a basis for a specification controlling the manufacture of pre-stressed girders. He acknowledged the debt due to Mr. T. Douglas and to the firm of Messrs. Braithwaite & Co. (Engineers) Ltd., for the development of that technique, which he considered to be as nearly perfect as it could well be made. He would like, however, to suggest that paragraph (14), *f*(v), of those notes, which dealt with the joints in the top chord, should be amplified to cover the provision of temporary brackets to support the chord in a horizontal position while splices were being made. The case of deck-spans might also be considered. The maximum wear allowed in bushes was given in paragraph (4) (*e*) as 0·008 inch, which compared with the slightly more severe 0·005 inch given on p. 115.§ The Author would like particularly to record his agreement with paragraph (14) (*j*) which stressed the importance of the uniform heating of rivets. In practice that was a most extraordinarily difficult thing to get done. There appeared to be something perverse in human nature which persistently made it heat the points of rivets more than the heads, and nothing short of a completely automatic heater would appear to be capable of overcoming that difficulty. He could endorse the statement made in paragraph (10). In the case of the Nerbudda bridge about 5,000 tons of steel had been fabricated before the first "trial" erection of a complete span was made, and that erection constituted span No. 16 in the bridge. Each span weighed about 680 tons.

With regard to the question of the loss of camber, which Mr. Everall had raised, it was recorded on p. 102 § that an initial loss of 0·11 inch had been measured when the span was carrying only a 120-ton locomotive which was caused to "slip." A further loss of camber of 0·42 inch occurred when the span was loaded with the full designed load. It appeared that top-chord-joint slip was the cause of both losses of camber, and that it therefore increased as the load increased. That appeared to preclude the possibility of slip occurring during and immediately after erection, as suggested by Mr. Everall. The initial stress imposed in members by pre-stressing had been everywhere of the same sign as that resulting from the application of a load, but as the load increased the stress would vanish and the

§ *Ibid.*

maximum primary stress in a given member was no more than that due to the designed load. It was possible that slip in the top chord had been due to the fact that when the chord-splices were being made, each section 35 feet 3 inches long was supported at the ends only and was in consequence sagging. The square butts could not therefore have been in perfect contact over the upper half of the section without drifting. The rivets in that region had therefore been closed with the two sections tending to move apart, and the direction of shear in them had been reversed when the compressive load had been applied to the chord. It was possible that that reversal might have accounted for the small amount of slip noticed. A splice in a tension-member would not be affected in the above manner. The Author.

The question of positioning the splices in the top chords away from the panel-points had already been referred to, and the change would tend to reduce the gaping of the square butts mentioned above. The use of straining devices to get the web-members correctly deformed had been mentioned again by Mr. Toms, but the difficulty of adequate supervision which that method involved ruled it out in the Author's opinion, except for special cases. A purchaser could never be sure of the extent to which they had been employed, and an alternative method which produced uniform results was far more satisfactory from that point of view. Mr. Toms' suggestion that the accuracy of alignment should be checked by scribed marks did not impress the Author. In the improbable event of inaccuracies of 0·001 inch or less being detected in that manner, Mr. Toms did not say how he would then proceed.

Mr. Toms gave three reasons why he did not favour the rotation of web-members as proposed in the Paper, but in stating his objections he had overlooked several important points. In the first place, dealing with members bent in simple curvature, he stated "... the components of the members had not only to be bent but had also, in most cases, to be axially stressed. ... The end rivet-groups would ... still have to do all the work of pre-stressing exactly as in the original method." That, of course, was not so. The type of deformation which was bound to occur was shown diagrammatically in Fig. 19, Plate 3 (following p. 160 §). In the case of a member 40 feet long and possessing a cross-section as shown in *Fig. 18* (*b*) (p. 119 §), each flange possessed an $\frac{L}{r}$ ratio of about 140, and an Euler crippling load when straight of about 200 tons.

§ *Ibid.*

The Author. The operation of pre-stressing to ± 2 tons per square inch bl drifting might be considered in two parts, firstly the rotation (the ends to a slope of about $\tan^{-1}\frac{1}{340}$, bending the flanges to a cir cular curve and causing a centre versine of 0·35 inch, and secondl the imposition of axial stress in the flanges. A pre-stress c ± 2 tons per square inch would involve an axial load of abou 65 tons in each flange if it retained its truly circular shape withou deformation during the operation. It might be shown, however that deformation was bound to take place, and such deformatio operated to reduce the nominal axial load of 65 tons. A reductio of from 30 to 40 per cent. could be expected in the case of the membe considered. That, however, was all that would be required, since as mentioned on p. 116 §, members bent in single curvature wer in any case easier to deal with than were those bent in doubl curvature.

In connexion with members bent in double curvature Mr. Tom stated that "it would be found that the amounts of the strains b which the lacing-bars had to be drifted would be so small that the would be inadequate for the production of strain in the mair section." Although the amount of fairing to be done in the case o the lacing-bars was small, it should be remembered that the amoun of fairing was the same for all the lacing-bars in the member, an that the loads on the drifts would be very light. In the case of th section shown in *Fig. 18* (*b*) (p. 119 §) of a member 40 feet lon bent in double curvature the amount of fairing at the ends of lacing bars 30 inches long would be about 0·0023 inch and would be th same for all. Alternately, the amount of fairing of the holes at th end of the member shown in *Fig. 18* (*a*) under similar condition would vary between zero and about 0·01 inch, depending on th position in the group, and the drifting would be heavy. The pro cedure mentioned on p. 117 § of applying a load to the span befor finally riveting the lacing-bar connexions was an alternative metho which should overcome Mr. Toms' objections, if they still remained

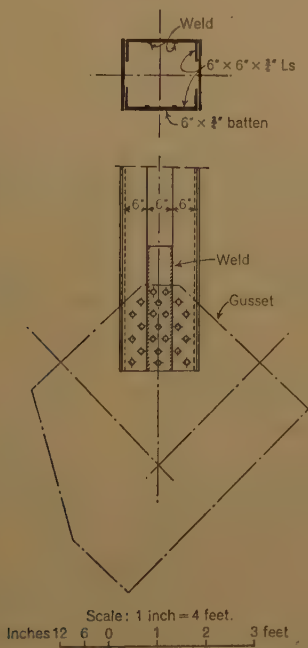
End connexions of members designed as in *Fig. 18* (*b*) (p. 119 §) had been found by the Author to offer no special difficulty; in fac they lent themselves to a particularly convenient form of joint It was shown in *Figs. 31*, and was self-explanatory. There coul be no objection to the use of welding in that way, since the batten plate could be extended so as to reduce the stress in the weld-fillet to any desired figure. That arrangement enabled a riveted joint t be made in the field, and there was no weakening (in the case o

§ *Ibid.*

nsion-members) by the deduction of rivet-holes. The weight of nsion-members was therefore reduced. The Author.

Turning to the flooring, the means of supporting the existing and e proposed types of flooring were shown in *Figs. 32* (*a*) and (*b*). was clear that the torsion in the supporting chord and the bending the web-members would be reduced, when troughing was used, an entirely negligible amount. The torsion would be very nearly

Figs. 31.

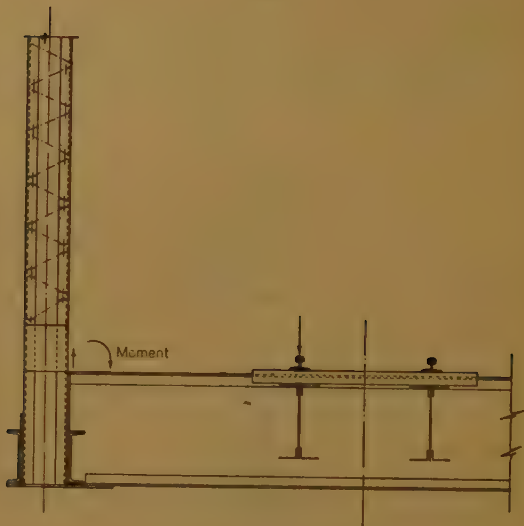


at due to the eccentricity *c* in *Fig. 32* (*b*), and the manner of taching the troughing precluded any appreciable end fixing moment be transmitted. With regard to the bending of the supporting ords by the troughing-reaction, to which both Mr. Everall and r. Toms had referred, since the supporting chord had been designed th that object in view Mr. Toms' objection to it was not easily derstood. When, however, the work of the stringers was performed by the chords the worst bending moment was reduced by e continuity of the chords from $\frac{WL}{8}$ to a figure which worked out

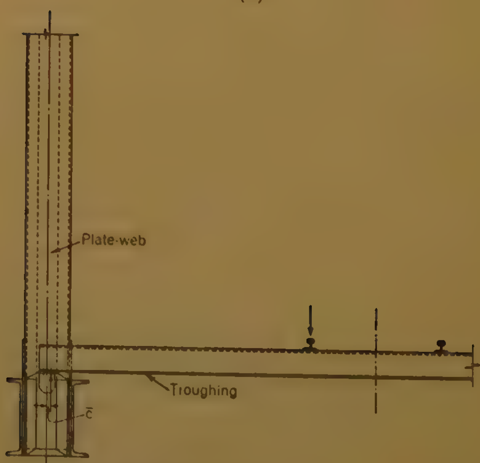
The Author.

Figs. 32.

(*a*)



(*b*)

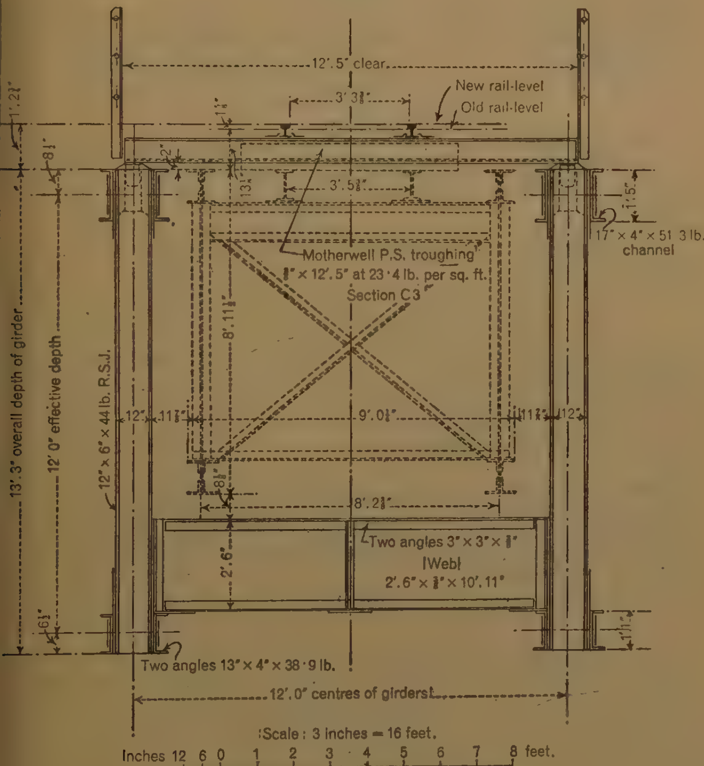


Comparison of Spans with Existing and Proposed Types of Flooring.

about $\frac{WL}{10}$ or slightly less, and a saving of metal was obtained. The Author.
rther, it would be found that the maximum bending moments curred at the panel-points, where relief was afforded by the esence of gussets.

Fig. 33 showed an actual example of bridge-girder renewal on

Fig. 33.



RENEWAL OF A BRIDGE-GIRDER.

gh piers (under traffic) in which the use of troughing in the manner dicated yielded substantial economic advantages from the point view of erection. In that case it was possible to erect the new an, with the exception of the trough floor, entirely outside the old an without in any way interfering with it. In order to transfer affic from the old to the new span it was only necessary to remove e old rails and timber sleepers and to follow up with the troughing

The Author. resting on the new top chords. The final rail-level was abl 1½ inch higher than the original. In reply to Mr. Everall's quest as to the savings likely to result from the employment of that t of floor, the Author would like to quote figures from two act designs. They were given in Table VII with current prices rul in Bombay.

TABLE VII.

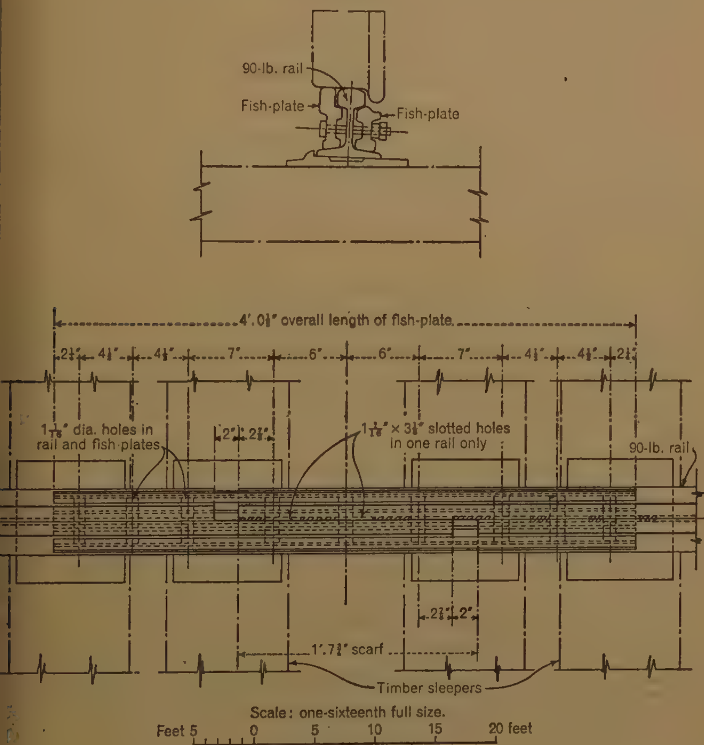
(1) Ravine viaduct No.1. Capacity: metre-gauge, single track, H.M. loading. Span: 105 feet; deck; structural steel.	(2) Mahi river bridge. 5-foot 6-inch gauge, single tr B+50 per cent. loading. Span: 220 feet; through; structu steel.
Steel in normal floor, 14 tons at Rs.310/- . =Rs.4,350/-	Steel in normal floor, 70 tons at Rs.310/- =Rs.21,70
As revised: Steel in troughing:— 8·8 tons at Rs.275/- . =Rs.2,420/- Extra steel in chords:— 3·4 tons at Rs.310/- . =Rs.1,055/-	As revised: Steel in troughing:— 59 tons at Rs.275/- =Rs.16,21 Extra steel in chords:— 14 tons at Rs.310/- =Rs. 4,34
Rs.3,475/- Less capitalized value of 70 timber sleepers replaced . . . =Rs.1,920/-	Rs.20,55 Less 110 timber sleepers=Rs. 6,55
Net cost . Rs.1,555/-	Net cost . Rs.14,00
Equivalent in tons of steel $=\frac{1,555}{310}$ . . =5 tons	Equivalent in tons of steel $=\frac{14,005}{310}$. =45·2 t
Steel in floor reduced from 14 tons to an equivalent of 5 tons.	Steel in floor reduced fron 70 tons an equivalent of 45·2 tons.

The saving amounted to about Rs.28/- per linear foot of t metre gauge span, and to about Rs.35/- per linear foot in the c of the 5-foot 6-inch gauge span. Those figures showed definite a substantial savings, but no account had been taken of the reducti of the secondary stresses and stresses due to lateral loads which t use of troughing ensured.

A rational manner of dealing with track-rails would appear to to provide an expansion-joint over each girder expansion-beari and to weld rails solid over each span rather than over two sp as suggested by Mr. Everall. The former plan was adopted in t case of the Nerbudda bridge. *Fig. 34* showed the type of rail exp sion-joint which the Author had designed for that purpose. T track had now been carrying traffic for about 2 years with compl satisfaction.

Mr. Toms had expressed doubt as to the possibility of measuring flexion-angles on the model. That operation offered no difficulty, [cut off]ause at the end of each member there was a solid piece of celluloid [cut off]resenting a gusset plate and upon that the scribed line remained [cut off]ight. In the case of the model used the straight portion was [cut off]m 1 to 2 inches long. For purposes of measuring angles a strip [cut off] celluloid about 15 inches long and 2 inches wide was used. A **The Author.**

Fig. 34.



Rail Expansion-Joint.

[cut off] was scribed down the centre of the strip and towards one end [cut off]er parallel lines 0·1 inch apart were also scribed across the whole [cut off]th of the strip. When the centre-line was laid over a member [cut off] made tangent to the left-hand gusset-line the displacement of [cut off] right-hand intersection could be read off immediately. The [cut off]et so read was divided by 100. The magnifying devices had [cut off]ked satisfactorily, and rotational slip at the point A in *Fig. 30* [cut off] not occur.

The Author.

Mr. La Touche's strain-gauge with a base of $\frac{E}{1,000}$ appeared offer advantages provided that the calibration of the instrum remained constant, and provided that the instrument was alw used on metal having a constant value of E. In the case of w on wrought-iron bridges, which constituted a considerable proport of the testing work in India, any advantage would be lost.

Full particulars relating to the Whittemore strain-gauge co be obtained from the makers.[1]

[1] The Southwork Foundry and Machine Co., Philadelphia, Pa.

CORRESPONDENCE
ON PAPERS PUBLISHED IN
MARCH 1937 JOURNAL.

Papers No. 5065 and 5097.[1]

"The Salonika Plain Reclamation-Works."

y Benjamin Williams Huntsman, B.Sc. (Eng.), M. Inst. C.E.

and

"The Lake Copais, Boeotia, Greece: Its Drainage and Development."

By Alec James Dean, B.Sc. (Eng.), Assoc. M. Inst. C.E.

Correspondence.

Mr. Oscar Borer observed, with regard to the Salonika Plain clamation-works, that Mr. Huntsman referred to the deposition of t consequent upon flood-discharges, and it was obvious that a eat deal of gauging had been done by the Foundation Company and predecessors to secure the figures which had been given in the per. It was, therefore, of particular interest later in the Paper see how far the designed velocities had been capable of carrying e silt in suspension. Mr. Huntsman showed that the ratio of the aximum flood-discharge to the carrying capacity of the channel nged from 9 : 1 to 7 : 1. The late Mr. E. C. Hillman, Assoc. M. st. C.E., who had been connected with river- and canal-work in ylon, had given a figure for English rivers of about 3 : 1, but it was mewhat difficult to arrive at a figure. From a consideration of the er Great Ouse, the ratio would appear to be between 2 : 1 and 1, but in the case of a river flowing through different classes of ad and material it became evident that the size of the normal er-channel and the flood channel was bound to depend upon the ntour of the land, the fall of the river, and the nature of the bed- terial. Where the bed was deep and the land adjoining the river s not unduly flooded, the ratio became only 2 : 1, whereas when the er was flowing through gravel with large stretches of land on either e at approximately bank-level, the ratio naturally increased con- erably ; as far as could be determined, however, the nature of the Mr. Borer.

[1] Journal Inst. C.E., vol. 5 (1936–37), pp. 243 and 287 (March, 1937).

Mr. Borer. land in Salonika and the Salonika Plain was fairly uniformly silt. would appear that the reason for the large ratio in the case of t rivers in the Salonika Plain lay in the figures given in Appendix V (p. 284 §), which showed rainfalls up to 6 inches per day (and ev then most of it fell within a few hours). Quite irrespective of t nature of the catchment-areas, which in that case were somewh mountainous, that would cause intense peaks in the flood-discharg That was also reflected in the run-off per square mile, which reach as much as 350 cusecs per square mile in one case, as compared wi a run-off for the Ouse and the Thames of about 8 cusecs per 1,0 acres.

The percolation-slope of 1 in 7 seemed to have been a safe on In the Fens it was mostly necessary to work to gradients of 1 in although in some cases it was possible to have a gradient of 1 in That could be improved by having berms at the back, but it wou be expensive on account of the claims for compensation which mig arise. In the same way the Foundation Company would appear have erred on the safe side with a freeboard of 1 metre as compar with the Fen allowance of 2 feet, which was the figure common used in England; then again, however, the sudden rise in the floo had to be taken into consideration which might equally easily exce the maximum which had been hitherto recorded. That was bor out by the record of sudden floods given on pp. 266 and 267 §, whi showed that a margin of safety, as it might be called, of 1 met had certainly to be provided under such flood-conditions.

The stripping of the base of grass before the embankments we built had been referred to by Mr. Huntsman; that was a practi which was followed on some of the rivers in Lancashire, the mater which was stripped off being afterwards used for protecting the n embankment. That might be an unnecessary refinement, howev in that past experience in the Fens seemed to have shown that ill effects followed the local custom of depositing straight on the gr after the removal of the heavy top growth.

The cross-sections of the river Axios showed that for convenience t excavation from the bed had been put on the berms. It would be interest to know the size of the silt-grains which were deposited unc those conditions, and whether there was any sign of the silt movi under flood-velocities of 4 feet per second, or whether it had beco consolidated and grown over, remaining in that position permanent The practice would lead to the danger of water lying at the foot the new embankment between that and the spoil where it could

§ Page numbers so marked refer to the Papers. (Journal Inst. C.E., vo (1936–37) (March, 1937).)—ACTING SEC. INST. C.E.

asily find its way back into the main channel, with the result that the main bank remained unduly wet, and might eventually lead to seepage into the external borrow-pits. Possibly the material of which those embankments were made was fairly sound, but experience in the Fens was against the practice of depositing dredged spoil on the edge of the washlands. Mr. Borer.

He was not quite clear as to the design of the Upper Notched weir, which seemed to be a satisfactory arrangement for dealing with the entrance of the flood-waters into the Circulatory canal. Apparently the waters passed through a series of V-notches, and the eddying consequent upon the change of velocity was dealt with by means of dry-stone pitching and curtain strips of stone bedding, placed right across the channel below at intervals of 4 metres. He could not quite follow the design of the apron below the notched weir; was there a direct vertical fall of about 3 metres on to the lower apron?

The provision of a breaching section such as that on the Moglenitsa was excellent practice, and it was of interest to learn that it was sufficient to make it of earthwork. The River Great Ouse Catchment Board had recently had to undertake the dredging and embanking of one of its Fen rivers, and they proposed to adopt such a spillway, as the holding-up of water to a higher level, consequent upon the construction of the embankment, would put such a strain on the banks that in the event of a burst there would be more danger than if the banks had not been so raised.

The constructional difficulties of dealing with the building of the new Circulatory canal while maintaining the existing rivers reflected great credit on the care exercised by the Company in the execution of the work. That was emphasized by the construction of temporary drainage-channels and by the general practice of excavating in a downstream direction so that the silt was carried downstream and the excavated section was left clear. That was contrary to general practice, but had been adopted by Mr. Borer when circumstances permitted, not only because the silt travelled downstream and could be picked up later, but also because the method was very valuable when dealing with weed-infested rivers; when work was carried out in the upstream direction the roots which broke off after the dragline had passed were carried down on to the new work, which formed an ideal bed into which to transplant them. The result was that the condition in a few years might be found to be worse than before the dredging had been undertaken.

He was interested to hear that wire-netting bags filled with stones had been used; such bags had been tried out on the New Zealand rivers some years ago, with great success. In view of the somewhat high velocities that had to be dealt with, would the Author state

Mr. Borer. whether subsequent experience had suggested that the groynes woul have been even more successful if they had been at any ang rather than at right angles?

There must have been great difficulty in some sections of the wo in maintaining "green" embankments, as owing to the great wid between the embankments considerable wave-action must have bee created by heavy winds under flood-conditions on account of th fetch. Experience in that matter was that the fresh embankmen that were most in danger were those which faced the stretches any particular river which was subject to the full force of the preva ing winds. One of the main barrier-banks in the Fens, which ha been raised and strengthened only a few years ago, had been subject to heavy erosion during the floods of the past year. Many years ag the Bedford Level Corporation had provided for the growth of osie beds immediately in front of the toe of the bank, so as to resist th action of the winds on the water, and that practice was much to b recommended, in spite of the possible loss of discharging capaci consequent upon the intrusion of the osiers; in his opinion it provid for lower velocities immediately alongside the embankment.

Mr. Huntsman stated that the spur-groynes were generally plac about six times their length apart, which was possibly about the usu practice under British conditions, although they appeared to be only one-half that distance apart in Continental practice. Th sentence at the foot of p. 272 § did not appear to be quite clea because if the spurs were spaced at three or four times their lengt apart that would imply that silt had accumulated right up to th front, whereas if they were further apart it would accumulate eve more. That, however, might be a misinterpretation of the statemen It was of interest to see Mr. Huntsman's conclusion that it wa cheaper to have spur-groynes than to employ continuous ban revetment.

There was much useful information in the evidence on p. 269 following upon the diversion of the Axios into its new chann Had Mr. Huntsman any information as to the size of the silt-grai which went into suspension, and the slopes of the sides which remain stable, after the river had bedded itself down? Was there an change in the coefficient of friction as the conditions change or was the alteration in surface-slope of the river entirely due to th change of cross-section? For long straight stretches such as exist in some of the works the bed should have remained level accordi to the observations elsewhere, and it was upon that fact that som of the theories with regard to the transport of silt had been base

§ *Ibid.*

It would therefore be of interest to learn whether the bed remained reasonably level across the channel. Mr. Borer.

Mr. Borer had endeavoured to compare the final velocities, as given on pp. 269 and 270 §, with those set out in the initial design. Without further particulars it was very difficult to compare them, but it would appear that the final velocities were somewhat lower than those for which provision had originally been made. At the same time the silt carried in suspension seemed to have fallen as the river settled down into its new bed, so that stability would appear to have been obtained with generally lower velocities; after all, all designs depended upon certain assumed coefficients, and the more information that could be obtained on that point the more valuable it proved on any future occasion.

Mr. FRANK GROVE, of Jersey, had been advisory engineer to the London Agency of the Foundation Company of New York from the inception of the Salonika Plain reclamation-works contract in 1925 until early in 1929. It had only been possible, in the time at his disposal early in 1925, to make a cursory examination of the scheme before negotiations for the contract were commenced. Those negotiations had been brought to a satisfactory conclusion in September of that year. He had been responsible for the original estimate attached to the contract of 1925, namely $26,570,000, on which to December, 1935, a saving of $10,910,000 had been effected. Mr. Grove.

His figure had been less than had been given in a report submitted to the Greek government by Mr. G. Yennidounias; that report had provided for the major works and had laid down the general principles which had finally been adopted. The very large saving had been due primarily to the excellent work done by the excavating and other plant, which had been selected solely on the advice and initiative of the contractors. It had been originally assumed that much manual labour would be insisted upon by the Ministry, but the officials had collaborated with the administration for the greatest possible economy. The saving was also largely due to meticulous care in the preparation of designs and estimates. It was greatly to the credit of the contractors and to the executive staff, bearing in mind the climatic and language difficulties, that those very heavy and intricate works had been carried to so successful a conclusion, and he hoped that most careful maintenance-works would be continuously undertaken so that the economic benefits to the district and to the finances of the Government would continue and increase. The contract had been on the terms of "cost plus commission." No other terms of contract had been possible at the time, the nature

§ *Ibid.*

Mr. Grove. of the work being so ill-defined until extensive surveys had bee completed and meteorological and other data had been procure over a period of some years. While such terms of contract wee readily sought by contractors, the administration and the executi engineers were frequently placed in an invidious position. The re master was the principal party to the contract—the Government. .

The first Agent of the contractors in 1925 had been Mr. B. T.] Boothby, M. Inst. C.E., who unfortunately after a few montl resigned owing to ill health. He was succeeded by Mr. Norm Sisson, M. Inst. C.E., who remained in charge until the early mont of 1929, by which time the surveys were completed, the major worl were designed, and the Ardzan and Amatovo lake reclamation wa well advanced to completion. Major W. J. Ross followed as Agen and during the 2½ years under his control the heavy works we commenced and substantially advanced. He was succeeded l Mr. K. J. C. Hill, B.Sc., M. Inst. C.E., and Mr. Huntsman took charg in 1934.

Mr. Lacey. Mr. J. M. Lacey observed, with regard to the Paper by M Huntsman, that the average slope of the Axios diversion-chann from the railway bridge to the sea was 0·0005, but at the commence ment it was about 0·00075 ; the new channel had been designed wit its bed- and water-slope parallel to the ground (p. 252 §). On pp. 26 and 270 § Mr. Huntsman stated that the river had been diverte into the new channel on the 5th April, 1934, the discharge in Apr being 6,000 cusecs, and the mean velocity in the upper reach on th 14th April being 4·75 feet per sec. The greatest bottom velocit observed had been 5·375 feet per second in April, and initiall the water-surface in the reach had had a slope of 0·00078 ; that wa the surface-slope had been greater than the bed-slope of 0·0007 Had Mr. Huntsman any records of velocity-observations taken a that time, showing the variation in velocity with the depth ?] was stated that after 8 weeks the surface-slope had flattened t 0·00075, and a year later for the same discharge to 0·00056, approx mately its average designed slope. In the meantime the mouth the river had extended seawards over 1,000 yards in a straight lir with parallel banks, so that apparently the river was adjusting itse to its regime-slope. As the mouth of the river proceeded seawar there would be a tendency for the bed to rise if the flood-water we controlled by levees.[1] Had the necessity of increasing the width the channel as it proceeded seaward been considered ?

§ *Ibid.*

[1] J. M. Lacey, "Some Problems connected with the Rivers and the Canals Southern India." Minutes of Proceedings Inst. C.E., vol. ccxvi (1922–2 Part II), p. 155 *et seq.*

Mr. Leitch.

Mr. W. O. Leitch considered that interesting maintenance-roblems would arise on the Salonika Plain reclamation-works. Damage might occur from maximum floods, during which the central hannel might deviate from a straight course and might attack the nprotected dikes, whilst serious erosion occurred in the high lands luring heavy rains. If the silt-proportion were too great for the urrent to carry, the space between the dikes would gradually ill up. Could Mr. Huntsman state the proportion of silt from amples of water taken during floods? The average proportion hroughout the year of 1 in 350 might not be applicable to flood onditions.

The space between dikes had to be wide enough to contain safely naximum floods, and smaller floods would therefore apparently be hallow, approximating to the conditions before the dikes had been nade, when the river-bed had become gradually raised above the djacent land. Was there any sign of that occurring?

Mr. Makins.

Mr. Thomas Makins observed, with reference to the lake Copais, hat the arrangement and extent of timbering required to ensure afety in deepening a tunnel 9·5 metres high through a stratum of issured limestone was bound to have presented very serious considerations. A travelling shield such as was generally used on the elining of railway tunnels might have proved of great service.

Had the rock-cutting in the tunnel been carried out entirely by he aid of pneumatic rock-drills and hand-tools? That question vas prompted from experience gained on extensive rock-excavation n the Piraeus district, 50 miles distant from lake Copais, where the eological formation consisted of calcareous marl and whitish limetone with numerous faults, fissures and cavities. It had been found here that the use of explosives invariably loosened more rock than vas desired. In the case of certain foundations which had to support articularly heavy loads, the necessary rock-excavation had been pecified to be carried out without the use of explosives. In consequence, the contractor's price for those excavations had been igher than the general price for unrestricted work. Notwithstandng that, however, when no one was looking, the temptation to blast ppeared to be too great, which illustrated the importance of constant nd careful supervision in that class of work.

Mr. Scott.

Mr. F. W. Scott, of Johannesburg, had found, when he had taken ver the Salonika Plain surveys early in 1926, that the work had been tarted on a basis of skeleton chain-traverses on which had been built p a tacheometrical system covering the area. He had continued hat system, although it had often occurred to him later that the onditions would have been better met by a triangulation-system. he survey-work of all the parties engaged in the area was reduced

Mr. Scott. as rapidly as possible to a common system of co-ordinates and datum levels, after which check-sights on well-known points were regularly picked up by the parties, and some of the advantages of a triangulation-system were retained. The gradients of the proposed canals were in many places so low that extreme accuracy of levelling was required, and the results reflected credit on the survey-parties, particularly when it was remembered that soft ground and swamps were the rule rather than the exception. Trouble arose in some cases as a result of the seasonal rising and falling of the ground-surface, and with it the bench-marks, due to the swelling of the ground in the winter rains. In that connexion he was gratified to note how closely the actual shrinkage of the mud in lake Yenitza corresponded with the prediction of 50 per cent. made in 1926 as the result of experiment. One matter had to his knowledge never been satisfactorily explained: the levels of the floor of lake Yenitza, taken and checked by most competent and reliable engineers, were found during construction to have risen by 1 metre, causing considerable difficulty in obtaining flotation for the dredger on the lake-drains. That that was the result of silt carried in by the Aliakmon in the 1928–29 floods seemed improbable, but it had never been explained otherwise.

The surface of lakes Amatovo, Yenitza, and the Kato Limni (the portion of the Loudias lying south-east of the Athens Railway) had originally been covered with masses of "sudd" (floating reed-beds about 2 feet thick), which gave trouble during both survey and construction. The edges of the lakes were also fringed with reeds, and in the case of lake Amatovo the burning-off of the reed-beds had lowered the ground surface by over 3 feet. He recollected an occasion on which he set up an instrument on the roof of a fisherman's hut in an island in lake Yenitza, only to find, on looking through the eyepiece, that the landscape appeared to be revolving slowly in front of him: the hut had been built on floating "sudd". The survey of the swamp-area offered many problems on account of its great extent. The use of three sections of 2-inch pipe, driven like a screw-pile into the lake-bottom and capped with a fixed table to hold the instrument, had been freely resorted to when sights from shore had become impossible.

The littoral drift off the mouth of the Loudias canal, mentioned by Mr. Huntsman, had been ascertained by the survey of floats connected to 4-gallon tins which had been sealed after being loaded with sufficient water just to submerge them. The tins were suspended just above the bottom, and the movement of the attached floats was taken as indicating the probable tidal current close to the bottom which could be taken into account in moving the silt laid down by the adjoining outfalls of the Aliakmon and Axios diversion. To avoid

he closing of the mouth of the Loudias by that silt, he had put Mr. Scott. orward proposals for submerged stone banks continuing the mouth of he Loudias seaward, to be extended as the adjoining silt deposits rew, but there was no record that the proposal had been adopted p to date.

It might be considered that the cost of the scheme was not justified y results, but it had to be remembered that Greece, a country omparatively poor in cultivable land, had been faced with a stagger- ng increase in refugee population, mainly agriculturists, for whom rovision had to be made no matter at what cost. Reduction of nalaria, too, had its value, although not easily priced in financial erms. Apart, however, from those questions, the port of Salonika, o important to Greece and to Jugo-Slavia, and such a factor in Near-Eastern politics, had been assured a long term of freedom from he fate of silting, which had overtaken so many ancient riverine orts in the Levant. He was whole-heartedly in favour of the alter- ative proposals put forward by the Foundation Company and by Messrs. Couper and Ker for a progressive series of diversions utilizing ulf areas near the port for silt-deposits, and laying aside funds for uture work on the main Axios diversion to accumulate at compound nterest. Theoretically that would have been less costly and would ave increased the life of the port still further, but the Greek Govern- nent took the unanswerable view that man could not foresee political onditions in 1977.

He would like to record his appreciation of the courtesy of the Hellenic State Railway engineers, whose co-operation in all matters ffecting their line, and especially their bridges, had been invaluable. He noted Mr. Huntsman's reference to wooden bridges over minor anals, and to steel superstructures for road bridges elsewhere, with nterest. In the earlier stages, the Greek Government were strongly pposed to the construction of steel or wooden bridges on the grounds f later maintenance. They pressed for reinforced concrete in all ases, and such bridges were built in some cases, where less expensive ridges would have fully met the case. It would seem that economy n first cost rose from a secondary to a primary consideration at a ater date. It would be noted that wherever possible, manually or utomatically operated sluice-gates had been avoided, and pre- erence given to self-acting foolproof structures. That was made a rinciple on account of the probable difficulty in finding suitable and onscientious attendants among a population quite unused to nechanical contrivances of any kind. The Lower Notched weir vas a special design, and was conceived on that principle. The great cale of the "notches" and the "no fall in bed" condition were quite unusual in a structure of that kind. The design was arrived

Mr. Scott.

at by two engineers working independently and by two separa methods, the results being practically identical. When the fir 1 : 100 scale model was put into operation it was found that t coefficient of the notch, normally about 0·67 in small notches, wa 1·00. The conditions of the experiment were not very favourab owing to the lack of co-operation by the local staff at the site, b there was no doubt about the discrepancy. Accordingly, a 1 : scale model was set up on the sea-front at Salonika, and suppli by pumping from the sea. A series of most careful observatio with varying rates of flow and varying upstream and downstrea levels was carried out; with duplication of the experiment on different scale, there could be no longer any doubt that the coefficie was as stated. The cause was undoubtedly the coffin-shape of t piers, which were lengthened downstream to resist the overturnin moment of the 14-foot 3-inch excess head anticipated on the upstrea side. The consequent "bellmouthing" had had the effect of increa ing the coefficient, and it was therefore possible considerably reduce the width of the structure. The model indicated that flow, after passing the notches, tended to rush straight down th central channel with excessive velocity, leaving dead water on th berms; that resulted in the introduction of the baffles mentioned Mr. Huntsman. It was a matter of interest that alteration in th position of the baffles in the model seemed to make no difference the position of the standing wave below the weir, nor was the ra of discharge affected. Advantage was taken of the model to hav similar experiments made for the Upper Notched weir. The curve form of the piers between the notches in the upper weir (shown *Figs. 5*. p. 254 §), resulted from the conditions of discharge. N single batter on the sides of the notches would effect the desir control at all heights of flow, and consequently an opening wit curved sides was required. It was a matter of great satisfaction Mr. Scott that those structures were said to have completely fulfill their purpose in the floods which had occurred since the completio of the works.

Excavation in considerable excess of the original estimates w reported by Mr. Huntsman. That was not unexpected in an ar nearly entirely composed of silt-deposits with extreme variation consistency. Experiments made by Mr. Scott had showed that so of the worst silt in a wet condition flowed to a slope of 1 in 6, b as drainage would effect a consolidation, it was impossible to desi for so flat a batter. Much of the excess of underwater excavati was bound, therefore, to be accounted for by the flow of mud in

§ *Ibid.*

he new channel, as well as by the inaccuracy due to working under- Mr. Scott. ater.

Much use was made of dragline scrapers working on pontoons s a result of the " sudd " in the submerged areas. Direct excavation y hydraulic dredger was impossible owing to the dense growth of egetation and roots. No machine requiring heavy anchorage ahead, uch as a bucket or shovel dredger, could be used, owing to the ifficulty of fixing anchors forward of the work in the " sudd." he very soft consistency of the bottom in most cases made spuds n uncertain proposition. The draglines, excavating towards the ontoon, could have their main anchors fixed behind them in the hannel already cleared, with light steadying anchors placed ahead or beam by the dragline itself, and for that reason they were adopted. hey had certain disadvantages: accurate grading of the bottom as most difficult, the output was from 20 to 25 per cent. less than hen working from land, and they could not be used for a final cut hen moving forward towards their work on account of the spill rom the buckets as they were dragged in, but that was overcome y taking a partial cut on the forward trip and finishing off the work o final level by a second cut while backing out. In the Kato Limni was arranged for the dragline to remove the top layer containing he reeds and other vegetation, and the final cut was made by the action-cutter dredger.

The extreme variations in the flow, particularly of the smaller vers such as the Gorgop (locally known as the " mad river " on ccount of its fierce and sudden floods) presented difficulties in he design of channels to meet as far as possible all circumstances. he Gorgop channel, for example, was excavated wide enough to eet flood-conditions, but during small flows there had been a strong ndency, while he had been on the works, for the stream to meander the wide new bed, cutting at the base of the bank and tending to rm loops. It would be of interest to know if it had been necessary revet that and other similar rivers, or if the trouble had ceased develop further.

Malaria had been one of the greatest difficulties met with in the rlier stages of the work, and the conditions of the Salonika Plain ad made it next to impossible to take anti-malarial measures during nstruction. In some parts the type of malaria had been malignant, d one refugee village had had to be evacuated after 110 deaths had curred in a fortnight, out of a total population of 225 families. At e start of the works on the Amatovo-Ardzan section, it was not common to have a sick roll of 20 per cent. during the autumn. egular prophylactic doses of quinine eventually reduced the pro- rtion to about 2 per cent. He observed that Mr. Huntsman

Mr. Scott. recorded the minimum temperature of the plain as 5° F. on the 20 December, 1925; during a Vardar gale in the winter of 1928– the Amatovo thermometers registered to −13° F.

Mr. Huntsman. Mr. HUNTSMAN, in reply to the Correspondence, considered th the ratio of the maximum flood-discharge of a river to the norm capacity of its natural channel depended not only on the contour the land, the fall of the river and the nature of its bed-material, stated by Mr. Borer, but also on its silt-content, and particularly its average discharge during the flood-season and the variation its discharge during the year. The following were figures for the tw largest rivers in the Salonika plain:—

River.	Axios.	Aliakmon
Capacity of natural channel: cusecs	14,700	15,10
Maximum flood-discharge: cusecs	134,000	116,50
Ratio	9·15 : 1	7·7 : 1
Average discharge (March and April): cusecs . . .	11,400	6,10

The Axios channel was comparatively more liable to siltatio but the Aliakmon floods were more rapid in their rise and fa (accounting for the average discharge in the flood-season bei comparatively lower), and were more violent in their scouri action. The ratio for stable rivers elsewhere might be 2 : 1 or 3 : but for torrential rivers much larger ratios would occur. The ne river diversion-channels were, in Mr. Huntsman's opinion, correct designed with capacities equal to those of their natural channel larger diversions would have silted, encouraging meandering of t low-water channel, and smaller diversions would either have cause more frequent inundation of the berms, interfering with wint grazing and summer cultivation, or else would have scoured, causi objectionable deposition of silt downstream. For the artific drainage-channels a ratio of 3 : 1 had been advocated (p. 248 §), a the Vardarovassi channel, for instance, had, as constructed, a rat of about 4 : 1.

Generally satisfactory results had been obtained as regards t transport of silt in channels where velocities had been designed the basis of the Kennedy formula $V_o = c.d^{0.64}$, using a coefficie of 0·84 (feet units), although in some cases a slightly larger val could probably have been adopted. In the Circulatory canal, whe the bed-material was coarser and firmer than the soils in the cent of the plain and where the silt was finer than the bed-material, t maximum velocity, 7·9 feet per second, had been designed grea

§ *Ibid.*

an the critical velocity, 7·3 feet per second; the results during Mr. Huntsman.
ods had been satisfactory as regards silt in suspension.

Mr. Lacey and Mr. Borer had asked for further information con-
rning the Axios diversion and the variation in its coefficient of
gosity. The following were figures concerning some of the measure-
ents taken on 31st May and 1st June, 1934, 8 weeks after opening :—

ilo-etre.	R: feet.	S.	V: feet per second.	f.	V_c: feet per second.	C.	N.	N_b.	N_c.
3	3·64	0·00079	3·04	1·40	2·60	1·17	0·0325	0·0244	1·33
6	4.34	0·00053	2·72	1·35	2·81	0·97	0·0339	0·0242	1·40
10	5·64	0·00027	2·73	1·30	3·11	0·88	0·0283	0·0240	1·18
13	5·32	0·00024	2·95	1·25	2·97	0·99	0·0238	0·0238	1·00
15	4·60	0·00015	3·13	1·15	2·65	1·18	0·0161	0·0233	0·69
18	6·63	0·00006	2·15	0·95	2·89	0·75	0·0189	0·0222	0·85

(Symbols explained in the succeeding text.)

t that time some scouring was still continuing in the upper reach
.3 and K.6). The section at K.15 was at the thickest part of the
lt which was being rolled along the bottom of the channel (p. 270 §).
he section at K.18 was further downstream than that rolled silt,
d there silt in suspension was being deposited. At K.13 the bed
d been raised during April with the material rolled down from the
per reach; some scouring of that material, which had commenced
the beginning of May, had ceased at the end of that month, and
the 1st June that section might be considered as having been in a
ate of temporary equilibrium, for the silting which occurred later
hen the discharge decreased did not commence until the end of
ne. The usual coefficient of rugosity, N, was deduced from Man-
ng's formula, $V = \frac{1 \cdot 486}{N} R^{\frac{2}{3}}.S^{\frac{1}{2}}$. That and other formulas in-
lving the coefficient of rugosity had often to be used in estimating
od-discharges at times when scouring or silting was occurring or
d recently occurred, and, as it was often difficult in such cases to
sign the correct value to the coefficient, its variation in the present
stance might be of interest. N might be considered as equal to
$_b \times N_c$, where N_b was a coefficient of "absolute" rugosity as
fined by the silt-factor,[1] and where N_c was a factor allowing for
e variation from a state of regime and measuring the channel-
ndition as opposed to the channel-material. No accurate analysis
the sizes of the silt-grains was made, but the silt-factor, f, had
en estimated on the following assumptions (made for the present
rpose of indicating the variation in N_c); (1) that at K.13, $N_b = N$,

§ *Ibid.*

Mr. Huntsman. since at that point the channel was straight, the cross-section w uniform, the bed and the silt in suspension were of similar mater and neither scouring nor silting was occurring at the time; a (2) that at other points the value of f was somewhat greater in t upper reaches and somewhat smaller in the lower reaches. V_c w the critical velocity as defined by $V_c = 1{\cdot}15\, f^{\frac{1}{2}}\, R^{\frac{1}{2}}$, and C was t ratio V/V_c. The amount of scouring which was occurring bel K.3 was approximately equal to the amount of silting which w occurring above K.18, so there was no appreciable difference in t silt-content of the water at those two sections. The figures indicat that there might be a 40-per-cent. increase over the normal value the coefficient in reaches where scouring was occurring, and that t coefficient might be 30 per cent. less than normal when the bed w composed of recently-deposited rolling material and when depositi of silt in suspension was in progress. The alteration in the val of N in the upper reaches after diversion was :

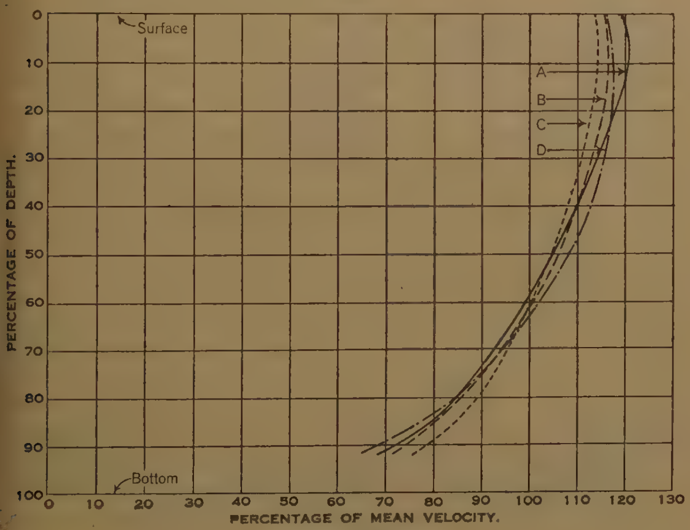
Date.	April, 1934.	June, 1934.	July, 1934.	Aug., 1934.	Sept., 1934.	Sept., 193
N :	0·0287	0·0317	0·0335	0·0275	0·0245	0·0255

Velocity-observations at various depths, about which Mr. Lac had made inquiry, hadshown great irregularity in the Axios diversio channel, as was to be expected in a channel adapting itself to new regime. Curves A, B and C (*Fig. 6*) showed the mean of observ tions in the upper reach of the diversion-channel in April 193 September 1934, and September 1935, respectively, and D was t usual curve for rivers in the Salonika plain, being the mean 140 readings on the Axios and the Aliakmon. Curve A was t flattest, that was to say, the rate of increase in velocity along t vertical was greatest, and the maximum velocity was nearest t surface, when the amount of scouring was greatest. Unfortunate no observations were available in the lower reaches where silti was taking place, but it was known that relatively greater botto velocities occurred there. The banks of the upper reaches beca steep and often vertical, as in the natural channels elsewhere in t plain, and the bed in the top reach did not remain level across b was generally deepest near one of the banks; in the lower reache however (for example at K.13 and K.15), the bed became fai level across with a slight rise towards the foot of each bank. T spoil from the channel, which had had to be placed on the adjace

[1] G. Lacey, "Uniform Flow in Alluvial Rivers and Canals." Minutes Proceedings Inst. C.E., vol. 237 (1933–34, Part 1), p. 421.

rms (p. 264 §), became consolidated. There was a smaller depth Mr. Huntsman. water in flood on that spoil than on the remainder of the berms, ld no direct scouring was occasioned by normal floods; trouble om scouring did occur, as anticipated, at points where the flood-ater on the berms flowed back into the central channel. There as considerable variation in the silt-content of the Axios during e year; as an instance, samples taken during a small flood in ctober, 1931, showed a mean value of 1 in 194 (by weight of dried t, excluding rolled silt), whereas 2 weeks later, at a low stage, e proportion had fallen to 1 in 1,180. The October observations

Fig. 6.



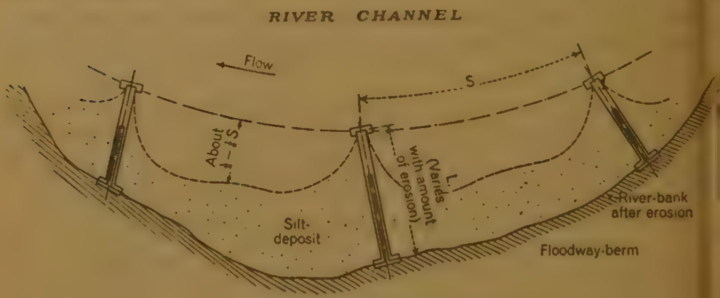
VELOCITY CURVES.

owed proportions of 1 in 255 near the surface and 1 in 156 at a int 1 foot above the bed. Mr. Leitch had referred to the well-own difficulties of maintenance of works like those of the Salonika ain, and to the tendency of the rivers' channels to deviate and ssibly eventually to endanger the protective embankments; ntinual inspection was necessary and the construction of river-aining works was required every year. The natural raising of the rms by silt and the consequent necessity of raising the crest-levels the embankments was not considered likely to be one of the most gent questions of maintenance. The problem of the mouth of the ios diversion, referred to by Mr. Lacey, would doubtless have

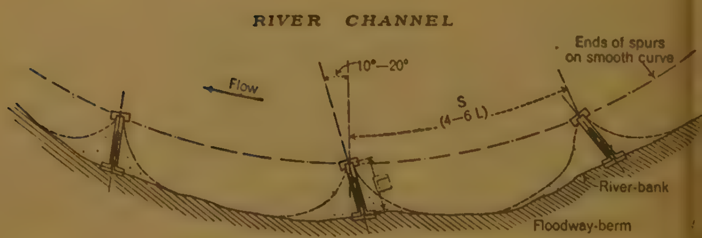
§ *Ibid.*

Mr. Huntsman. been solved more satisfactorily by the scheme for a progressive se of diversions with their embankments widely bell-mouthed (p. 252 but, as Mr. Scott had mentioned, the Government had had to g consideration to other views besides those of engineering. proposed stone dikes at the mouth of the Loudias canal had yet been built, as the authorities had preferred, before committ

Fig. 7.



SPURS FOR RECLAIMING ERODED RIVER-BANK.



TRAINING SPURS, FOR RIVER-BANK PROTECTION.

(Not to scale)

SPURS IN THE RIVER AXIOS.

themselves to any expenditure, to wait for the action of time indicate their necessity.

With regard to the spacing of spurs in the bed of the river Ax Mr. Huntsman hoped that the sketch (*Figs.* 7) would supply information which Mr. Borer had asked for in addition to the n given on p. 272 §. The outer ends of the spurs were designed on easy curve, and the deep-water channel had to be left wide eno to maintain a good discharge-capacity. The use of wire-nett

§ *Ibid.*

ags had been common for some years in Italy and Switzerland. Mr. Huntsman.
he life of the bags depended upon the efficiency of the galvanizing, ad severe testing of that galvanizing had been included in the pecifications.

Mr. Scott had remarked upon the difficulties in the design of hannels where there were extreme variations in the flow. The mount of maintenance-work which had been required in the Gorgop, hich he had mentioned, had not been very great; the channel had aintained its alignment, but the clearing of silt-deposits had been ecessary in the lowest reach. There had been, of course, a tendency several cases for streams to meander in their channels at low ages, but bank-pitching had not usually been needed; those oubles which had developed had generally been due to excessive adients. A certain amount of "bed-heave" had occurred in hannels where the spoil had been deposited close to the edge of the it, and in the Circulatory canal a small rise and fall of the bed had een observed, varying with the rise and fall of the water-level. ne theory to account for the observed rise of part of the central ed of lake Yenitsa had been that the rise resulted from the serious rthquakes which occurred after the surveys had been completed. r. Scott had commented on the experience of draglines mounted pontoons and using drag-buckets; actually most of the work by aglines in lake Yenitsa was done with grab-buckets. Better results ad been obtained with grabs, as had already been mentioned p. 262 and 315 §).

Stripping the sites of embankments had been necessary, and key-enches also had been required in some instances; moreover, that ripping had provided the top-soil which was needed if grass were be successfully grown on the slopes, and the stripping of the orrow-pits was essential, he considered, in order to exclude roots nd vegetable matter from the core of the embankment. Stone-tching had been laid on embankments exposed to wave-action; scouring were likely a strong foundation or flexible apron had to e provided at the toe of the embankment. On other works in orthern Greece thin "gunite" work had been used on the slopes consolidated embankments; this had appeared to be quite tisfactory, but in the Salonika plain the ordinary stone pitching ould be done very cheaply.

Mr. Borer had inquired whether there was a direct vertical fall about 3 metres at the Upper Notched weir. Reference to *Figs. 5* . 254 §) would show that the fall in the bed-level was dimensioned 2 metres: that was correct, the upstream and downstream bed-

§ *Ibid.*

Mr. Huntsman. levels being 22·17 and 20·17. As the water-level downstream i the canal rose more quickly than the water-level upstream in t gathering-basin, the fall in water-level at the weir became l than 2 metres as the flow increased, and at high-floods level t difference in water-level was 40 centimetres, as was shown in *Figs.* The curved tortoise-shaped sides of each notch (*Fig. 1*, facing p. 306 automatically maintained the correct levels of the upstream a downstream water for all discharges up to 8,500 cusecs ; above th level (25·17) " backing-up " resulted upstream as previously d cribed (p. 255 §).

The temperatures recorded in the Paper (Appendix I, p. 277 were those officially measured at stations in the Salonika plai lower minimum temperatures had certainly occurred elsewhere the district, and higher maxima also had been unofficially report Mr. Grove had emphasized the importance, in the case of works li those in the Salonika plain, of the correct choice of plant ; th would be appreciated, since 1 penny extra per cubic yard on t cost of excavation by machines would have increased the cost over £200,000. Reference had been made to the estimated costs bridges in the choice of the types to be adopted. Excluding so of the earlier small bridges, which had been relatively expensive, t actual costs of some typical road-bridges constructed between 19 and 1935 had compared as follows :—

	Type.	Name.	Cost per linear foot.	Cost pe square fo
Main roads:	Steel (short)	Loudias	283	14·4
	Steel (long)	Axios	158	8·0
	Reinforced concrete	Vardarovassi	118	5·7
Minor roads:	Reinforced concrete	Selimli	47	3·6
	Reinforced concrete and timber	Koulakia	42	3·5
	Timber	Koufalia	26	2·2

The above figures included an allowance for overhead charges, not for the cost of land, etc. Details of those and other brid had been given in Appendix VI (p. 282 §). Timber bridges for mi roads had often been adopted on account of their low first cost, bridges of reinforced concrete had been constructed very chea and their extra cost was often justified on the grounds of chea maintenance ; timber bridges had a comparatively short l particularly as there was often a scarcity of firewood in adjac villages.

§ *Ibid.*

Reference had been made also to the cost of the whole scheme. **Mr. Huntsman.** t was not possible to make an exact sub-division on the total cost, ut approximately $2,300,000 had been spent on works of com-iunication and on bridges, and about $1,400,000 on diverting the ver Axios. That left $12,000,000 as the cost of draining 108,000 cres and of protecting from floods a further 198,000 acres, giving n overall rate of about £8 per acre, which seemed a reasonable gure. In 1928 Greece had had to import 2,950 million drachmae ibout £5,500,000) worth of agricultural products, including 525,000 ons of wheat. The extent to which production had since been nproved could be judged by the following figures :—

Year:		1928.	1934.
otal cultivated area : square kilometres	Whole of Greece	15,901	21,445
	Salonika Dept.	1,459	2,356
ereal crops: tons	Whole of Greece	785,842	1,302,095
	Salonika Dept.	—	215,365
alue of cereal crops: million drachmae	Whole of Greece	3,535	5,565
	Salonika Dept.	—	891

Mr. DEAN, in reply to the Discussion and Correspondence, observed **Mr. Dean.** iat the *katavothras* in the Copais and the surrounding districts eemed to have aroused interest from time immemorial. In recent mes they had been studied by Dr. A. Phillipson[1] and by Mr. . S. Cole, Assoc. M. Inst. C.E., in 1910, when Chief Engineer to ie Company. Although they appeared to have been used by the ncient inhabitants for drainage purposes and similar use had again een proposed in recent times, their existence was by no means an nmixed blessing. Without those subterranean outlets it was to be resumed that the lake would have risen high enough for the water) cut out for itself an overland channel to the sea, and it might in me have drained itself naturally.

Most of the water from the *katavothras* would appear to go direct) the sea ; springs of fresh water appeared along the whole coast pposite, near the edge and in the bed of the sea. A particularly rge spring appeared at Skroponeri in line with the Grand *katavothra.* he use of the waters from the Copais for hydro-electric purposes ; Anthedon was also rendered speculative because of the leakage om the lakes Likeri and Paralimni through *katavothras.* In con-exion with that project the Author attempted, in 1931, to locate ro *katavothras,* of which there was some record, in lake Paralimni. ne had been eventually found some 20 metres below the surface

[1] *Zeitschrift der Gesellschaft für Erdkunde zu Berlin,* vol. xxix (1894), No. 1.

Mr. Dean. of the lake, the flow of water being clearly indicated by a curren meter. Several buckets full of a strong solution of fluorescein ha been poured down a pipe at the spot, and as no sign was observe at the surface, presumably it had all gone into the *katavothra*, bu an elaborate picket of all suspected points on the seashore ha failed to observe any sign of coloration. It might be said tha much smaller doses of fluorescein put into the water upstream o Anthedon tunnel, besides colouring the whole stream, were visibl hundreds of yards out to sea. It would seem that besides *kata-vothras* which were large enough to be noticed there were probabl hundreds of smaller ones, and that they were all probably inter connected below ground, so that the water took a long time to reac the sea.

The scheme adopted of deepening the outlet-works in order t improve the drainage of the lake would certainly seem to have bee the right solution. It was now proposed to complete the lining o the tunnel and to render the bed and sides of the Emissary cana with cement mortar in order further to improve the drainage fro the Great Central drain (thus combining the scheme adopted wit the alternative mentioned by Mr. Halcrow). The lining of th tunnel was, however, primarily intended to ensure its safety. A Mr. Makins observed, considerable care was required in deepenin a tunnel of that size. Explosives were employed in restricte lengths but, as mentioned in the Paper, a serious accident occurre probably through the contractor's desire to go too fast, althoug there was no evidence that blasting had occurred where instructio were to the contrary.

As mentioned in the Paper, little silting or scour appeared to hav occurred in the lower channel of the Grand canal, whereas siltin of the berms was in places considerable, particularly at the bend A survey was in hand to determine the precise position. He though that Mr. Borer was probably right in assuming that a self-cleansin velocity kept the central channel clear in flood-time; the velocit on the berms was less, and there silting took place. In normal co ditions (that was to say, when the water was confined to the low channel) the quantity of silt was negligible. In its upper reaches th channel had been excavated mostly in the heavy alluvial soil of th Kefissos and Hercyna deltas, but further east the sub-soil of cl rose nearer and nearer to the surface. In the latter soil artifici attempts to encourage scour had not proved very successful. Th increases in the discharges of the rivers flowing into the Grand can as shown in Tables II and IV (pp. 293 and 304 §), compared wi

§ *Ibid.*

able I (p. 291 §), were due to further observations since Monsieur ochet had made his original estimates. Mr. Dean.

The sinking of the old lake-bed described in the Paper (p. 293 §) as almost entirely due to the burning of the peat. When the nterior canal had been designed it flowed through the lowest land. fter the peat had burnt, the land in the centre of the Copais sank places as much as 4 metres, and the area marked on Fig. 3, Plate 1 ollowing p. 316 §) became the lowest land at a level more than metres lower than the previous lowest level, but not in the same lace. That would explain to Mr. Borer the difference between the aximum shrinkage of 4 metres and the fall in level of the lowest nd of only about 2 metres. Dust storms in the summer were bound remove a certain amount of soil from the Copais, but no investiga- on as to the quantity had been made.

The only channels which were embanked for drainage purposes the Copais were the Grand and Marsh canals, and the Melas river nd canal. Apart from very short floods from torrents the Melas ver had no flood-discharge to carry and, since it had been deepened, e water-level was normally below the level of the land and generally elow the burnt-peat layer, so that infiltration from it had practically opped. The floods which interrupted gravitational drainage were the Grand and Marsh canals. When the water-levels in those anals were too high the regulators at the outlets of the Melas and terior canals and the Great Central drain were closed. Actually e Melas and Interior canal regulators were scarcely ever shut, but was when the Great Central drain regulator was closed that the eading-up of the drainage-water from it tended to cause flooding the lowest land. Paradoxically, it was not the maximum flood- vels which were a danger to the crops. Those floods were of short uration. A minor flood-level, which was liable to occur in February d March, due to a combination of rain and melting snow off the ountains, and to last for a long time, might interrupt the flow of e Great Central drain for days and cause flooding of young crops. was then that pumping had to be resorted to (p. 300 §).

The drainage-improvement works, combined with the siphon der the Marsh canal, had solved the drainage problem of Division A" (Fig. 3, Plate 1). Experience as to warping the basin was still nall. Water was let into the basin through sluice-gates in the arsh canal bank, and it was hoped in time to improve the quality the land by silting. The valuable floods for that purpose were the st floods of early winter, which were not usually high enough to ood much of the basin. The construction of a barrage in the Marsh

§ *Ibid.*

Mr. Dean.

canal, suitable for use in winter, was being considered. Examinatio of flood-water had shown a silt-content as high as 1 part of dry s to 1,600 parts of water, by weight.

The Emissary canal was pitched with dry stone about 30 cent metres thick throughout its length, except in the deepest part, length of 800 metres at the downstream end. There masonry cement mortar 50 centimetres thick replaced the dry stone pitchin in alternate panels about 3 metres wide. No serious trouble h ever occurred in the canal upstream of that length. The stretch question was through a soil changing from gravel at the top throug alluvium to clay at the bottom. Since the deepening had bee completed no sign of slipping, such as occurred before, had bee observed.

When it was a lake, and during the early years of the drainag works, the Copais had an exceptionally bad reputation for malari To-day, although the reputation still held to a certain extent, malar had ceased to be a serious trouble. As a matter of interest, wit reference to Mr. Huntsman's and Mr. Scott's figures for the Salonik Plain, the maximum and minimum shade temperatures recorde in the Copais in 30 years were 113·4° F. in 1916 and 5·0° F. in 19 respectively.

Paper No. 5101.[1]

"Fundamental Research on the Application of Vibration to the Pre-Casting of Concrete."

By Donald Arnott Stewart, A.M.I.E.E.

Correspondence.

Mr. Broomfield.

Mr. R. E. Broomfield, of Lagos, Nigeria, observed that som people might think that the research as so far conducted migh have rather too little reference to actual practice for it to be applie to actual conditions. It was clear that the properties of concre as a function of "statical" variants, such as water-conten aggregates, etc., would scarcely depend at all on the size or shape the mould, and thus laboratory experiments could lead directl to commercial application. Considered, however, as a function some mechanical movements, that was, "dynamically," the sha

[1] Journal Inst. C.E., vol. 5 (1936–37), p. 318. (March, 1937.)

ıd mass would appear to be of considerable effect, and he would be terested to hear that some attempt had been made to consider ıat point. It would appear that what might be the best frequencies ıd magnitudes for a 4-inch cube of close-grained concrete might ear little relation to that needed to produce the same effects on a inch flat-slab floor, or on a 3-foot-deep beam with heavy inforcement. Mr. Broomfield.

Dr. E. J. HAMLIN, of Johannesburg, had tried out certain patented ommercial vibrators to consolidate the concrete in a reinforced-oncrete sewage-storage tank 20 feet by 20 feet by 15 feet high. he vibrators had been stated to have an effective range of approxi-ately 54 inches diameter with the vibrator as centre. On the lvice of the suppliers four machines had been used. The concrete roportions by volume were 1 : 2 : 4, with a slump of 2 inches. He as not in a position to state the frequency of the vibrators. When e formwork had been stripped the appearance of the concrete had een very disappointing, portions having been very good, whilst ljacent to such good portions there had been patches where little onsolidation appeared to have taken place and the concrete was orous. Dr. Hamlin.

He thought that the following might be an explanation of the ısatisfactory results obtained. The energy-waves set up by the brators diverged in all directions in the plane of the formwork, milar to the ripples formed on the surface of calm water in which a ebble was dropped. Where there were two or more adjacent urces of exactly similar waves spreading in a plane, there were ound to be places where the crests (or troughs) of one system incided with the crests (or troughs) of the other. At such places ere would be waves with unusually high crests (or deep troughs) ıd violent agitation in the concrete. At other places, again, the oughs of one system might coincide with the crests of the other and l disturbances might be obliterated. At such places little disturb-ıce in the concrete would result, with consequent poor consolidation.

Whilst admitting the advantages established for vibrated concrete, would like to know whether the Author considered that similar ıiformity could be obtained when dealing with large volumes of ncrete, and also whether reinforcement would have any effect in nsolidation by means of vibration. Further, had the Author taken res from the vibrated specimens to ascertain whether the density ried with the distance from the source of vibration?

Mr. G. P. MANNING pointed out that the Author had defined the rm "amplitude," but had omitted to define what he meant by frequency." Such a definition was urgently required. He had ırchased a number of vibrators operated by an eccentrically-loaded Mr. Manning.

Mr. Manning. rotor. The machines rotated at a speed of 2,800 revolutions per min and according to his definition the frequency of vibration was the fore 2,800 per minute. The makers, however, claimed that machines gave a positive and a negative impulse each revoluti and that they therefore had a frequency of 5,600 per minute.

The definitions of the terms "amplitude" and "frequenc should be agreed upon, and should be made known to all concerne

Mr. Marks. Mr. J. R. Marks, of Auckland, N.Z., observed that the Autho research had been confined to the application of external vibratio (that was, vibration of the whole mould), and that therefore the effe brought out would be helpful mainly to a consideration of t moulding of pre-cast concrete products by vibration (as implied the title of the Paper), rather than to concrete construction *in si* which necessarily involved quite different treatment.

In considering the results of the experiments the particulars of t mix the Author had employed should not be forgotten, as he had ve properly indicated. For instance, he had used ⅜-inch maximu aggregate, of angular shape, in what was bound to be regarded an extremely lean mix for that maximum size of stone. Clea there was no surplus of cement-paste at all over voids in the d rodded volume of mixed aggregate, whilst the mix, being far fr plastic, could have been hand-placed only by a great amount of ha ramming, so as to pack the aggregate tightly enough to enable t cement-paste approximately to fill the voids.

Mr. Marks thought that it would be helpful to those studying t Paper if more particulars were given of the mix, such as, for instan an expression of the mix by dry-rodded volumes of sand and sto respectively, to 1 cubic foot (94 lbs.) of dry cement, and also of t "real mix" by volume of dry-rodded mixed aggregate to the sa unit of cement. He estimated the latter to have been about 1 : but he had had to make some "scientific guesses" to arrive at th approximation. It would also be of interest to know the respect fineness-moduli of the sand and stone, and their apparent absol densities. What was the true water/cement ratio of the mix, weight or by volume, after allowing for absorption in the aggreg particles?

In connexion with *Fig. 4* (p. 326 §), would the Author indicate t comparative density of the hand-packed specimens? It wo appear to have been about 146½ lbs. per cubic foot. He notic that the density curve in *Fig. 4* (p. 326 §), for 6,000 vibrations, agre with neither that for the first nor the second series in *Fig. 6* (p. 328 nor was it an average of those series. Again, the strength-curve

§ Page numbers so marked refer to the Paper. (Journal Inst. C.E., vo (1936–37), (March, 1937).)—Acting Sec. Inst. C.E.

g. 5 (p. 327 §), for 6,000 vibrations, was different from that given Mr. Marks.
Fig. 6 for the first series. Had there been a further series, or if ot, what series was represented in *Figs. 4* and *5?*

Referring to the question of densities (*Fig. 4*) as compared to the obable hand-packed density, it would seem that, at an acceleration 3*g* and upwards, both frequencies of 3,000 and 6,000 vibrations per inute developed small increases, whilst 12,000 vibrations per minute pparently failed to reach the hand-packed density at an acceleration low about 7*g*. In the latter cases it would appear probable that the olume of cement-paste might have been inadequate to fill the voids, articularly at 3*g*. Could the Author throw some light upon that uestion? Might it not be that the mix was just a little too lean give fairly comparative results in the whole set of experiments? any case, Mr. Marks suggested that it would be too lean a mix for ny manufacturer to risk in the practical moulding of concrete oducts.

It would also be most instructive to know if the curves for the ree respective frequencies would fall in the same order if larger aximum stone (say ¾-inch) were used instead of ⅜-inch. It would e interesting to ascertain what actually happened in the cement-aste itself through high-frequency vibration. The closer compaction the aggregate particles and the occurrence of the three stages bserved by the Author (even if only water but no cement were esent), could be readily understood. That was bound to reduce e voids requiring to be filled by cement-paste to a small extent. e could also understand that suitable vibration would permit the acing of a mix at very much lower water/cement ratio than would e practicable with hand-placing for the same cement-content. he former would be a factor of economical value only, and a comparatively minor one at that, as indicated by the small differences density of the concrete. The latter factor (drier mixes) would ppear to be the one from which most of the benefit of vibration me. For a given cement-content it was bound to result in stronger ncrete, merely by virtue of its permitting the adoption of a lower ater/cement ratio in the first place. Alternatively, for a given rength, it was bound to represent an economic factor of considerable lue, because it was not necessary to waste cement by providing ore cement-paste (of the consistency necessary for that given rength), merely to provide the "workability" factor.

Those views were based upon the assumption that the strength concrete depended solely upon the quality (not to the quantity) its binding cement-paste (that was to say, upon the water solidus

§ *Ibid.*

Mr. Marks. cement ratio adopted at the time of mixing), provided that the voi in the aggregate were completely filled by such paste. In other word the strength of concrete was related to the density of its harden cement-paste, and not to the weight per cubic foot of the actu concrete. In fact, lean weak mixes of hand-placed concrete we normally heavier than rich strong ones, owing to the presence of mo aggregate solids, which were normally from 60 to 100 per cen heavier than hardened cement-pastes.

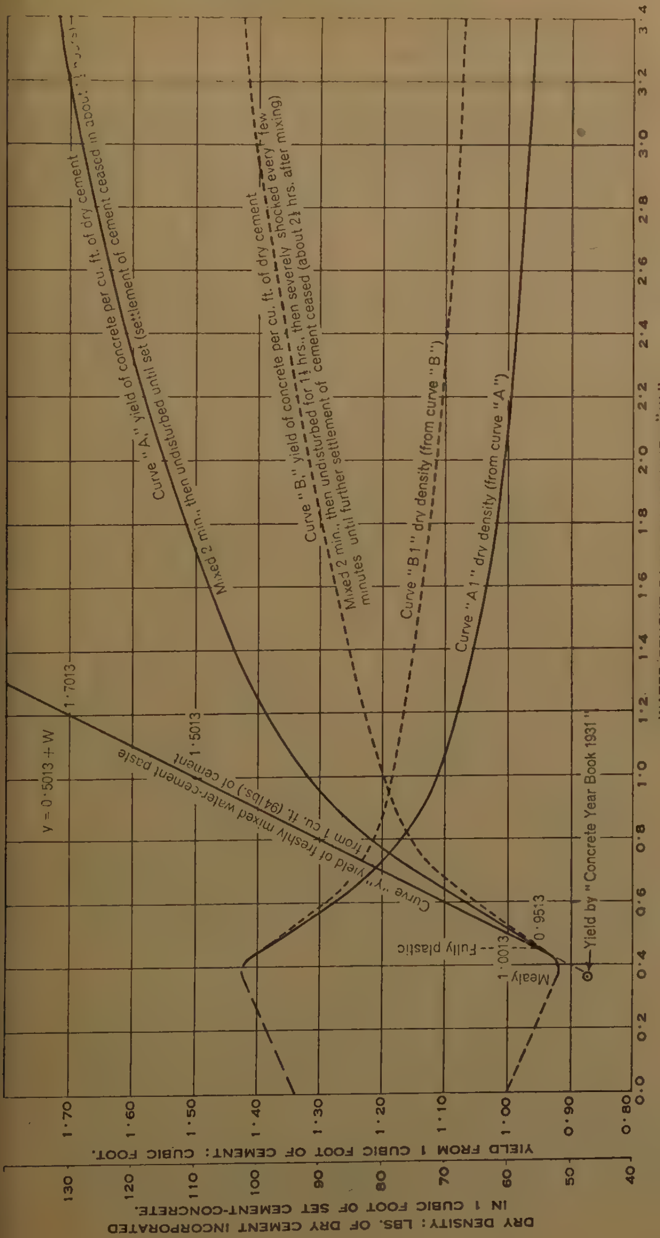
There was no doubt that, from the above considerations, vibrato placing (where such could be employed with certainty of n extraneous or secondary detriments) was bound to have a decide economic value, while, in addition, it reduced shrinkage and crazin defects. Apart, however, from the undoubted benefits to whic he had referred above, he would like to know if vibratory mouldin improved the resultant strength of a given cement-paste or otherwis That was to say, did it actually increase the density of the harden cement-paste ?

The increased concrete-densities shown in *Fig. 4* (p. 326 §) we inconclusive on that point, partly because the mix was so lean th they might, perhaps, be attributed to closer packing of the aggrega alone, with the consequent exclusion of more air-pockets and fr water-pockets and even the exclusion of more cement-past Further, the exclusion of more air- and water-pockets could easil account for the increased strengths shown in *Figs. 5* and *9* (pp. 3 and 331 §). It should be remembered that all cement-paste exude some excess water during setting, even if allowed to lie perfectl still after placing. That water either found its way to the surfac or else became entrapped as globules, ultimately leaving cavitie In either case, however, it definitely separated from the actu hardened paste. The question was, therefore, did vibratory placin actually increase the amount of excess water separating from th cement-paste, or did it only (or even) accelerate the time of its bein freed ? The interesting fact that the Author had encountered a apparent initial set in his endeavour to find a "rigidity-factor might suggest that either one or other (or both) of those effects wa brought about. Alternatively, was it possible that a "flash set was developed by heat-generation caused by the vibrations ? Th latter question seemed well worth investigation. The energy th could create the explosive forces he described in the stage "redistribution" was bound to be very considerable.

Mr. Marks considered that some tests of high-frequency vibratio upon neat-cement mixes at different water/cement ratios would

§ *Ibid.*

Mr. Marks.



Variation in Dry Density and Yield with Water/Cement Ratio for "Star" Brand Cement.

Mr. Marks. a useful contribution to the subject, and he suggested that the Auth might undertake such tests.

In 1931 Mr. Marks had made a series of tests upon neat cemen from which *Fig. 15* had been compiled, showing the yields of fresh mixed and hardened cement-pastes at different water/cement ratio with a normal Portland cement. The set-yields had been teste respectively for undisturbed pastes and for those which had bee subjected to late vibration, in the form of tapping the table upo which the containers rested. Those tests had brought out th following apparent facts :—

(A) That, if undisturbed, the cement ceased giving off exce water after approximately 1½ hour (about 1¼ hour fo rapid-hardening cement), for all water/cement ratios.

(B) That early vibration of the form adopted (namely, extreme low frequency but fairly high acceleration) did not appea to hasten the rate at which excess water separated, or t increase the total water given off.

(C) That, if vibration were commenced at the age of 1½ hour, further separation of excess water occurred, resulting much denser and proportionately stronger cement-pas when hardened.

The densities shown in *Fig. 15* represented the weights of d cement that became incorporated in each cubic foot of harden cement-paste. To obtain the approximate actual dried-out weigh per cubic foot, about 20 per cent. should be added for water pe manently retained in chemical combination. Tests made later concrete mixes in test-cylinders and in slabs showed a gain strength of about 20 per cent. through similar late vibration.

In the United States the method of concrete-road constructio known as the "Johnson Method," in effect adopted that practic with the result that independent and authoritative tests indicat conclusively that 94 lbs. of cement was saved per cubic yard, witho reduction in strength.

He referred to those matters, not with regard to the late vibrati of pre-cast concrete, but to draw attention to the apparent fact th there was a critical age before which cement-paste might not tightened by slow vibrations. It might be that high-frequen vibration reduced that age ; otherwise the conclusion would have be reached that it did not tighten the cement-paste, but only p mitted the mechanical packing of a drier-mixed concrete.

Mr. Mitchell. Mr. JAMES MITCHELL observed that on p. 318 § the Author stat

§ *Ibid.*

Mr. Mitchell.

ıat, as the result of vertical vibration, the force of gravity was eriodically neutralized, and that during those neutral periods the articles of the concrete " tend to twist and turn." There was no ue offered, however, to the nature of the force which produced ıat twisting and turning. Anyone who had observed the effect f striking the shuttering of a mass of concrete horizontally with heavy hammer, would probably be satisfied that the direction of ıe vibration had comparatively little to do with the resulting conolidation of the concrete. The effect of the vibration was to set ıe particles in motion, and thus enable gravity to move the solids ownward, while at the same time the upward expulsion of air and xcess water was facilitated. On p. 346 § reference was made to the uestion of whether workability or water-content was the best basis f comparison between concretes. It could not be too strongly mphasized that on the works workability was the more important, ater-content being a means toward that end.

On p. 334 § it was stated that " a casting which has been made by ibration is a system of particles precisely adjusted." No system of ımming could produce such a precise adjustment, and the difference etween vibration and hand or other ramming was one of degree, ıd not of kind. It was impossible in practice to prevent a certain mount of segregation, by " segregation " being meant the uneven istribution of the constituent parts throughout the mass of the oncrete. Merely churning the materials together in a mixer did not ısure that each particle would occupy the most desirable position ı the mass. Even if it did, the subsequent operations, before the oncrete reached its final position, would disturb any such nice alance. In order to counterbalance that segregation, it was ecessary in the mortar to provide a proportion of cement in excess f that just sufficient to fill the voids in the sand, and in the concrete provide a proportion of mortar in excess of that just sufficient to ll the voids in the stone. Furthermore, in order to produce a roper degree of workability, it was necessary to provide a proportion f water in excess of that just sufficient for the proper setting of the ement. One of the most important functions of the water was to duce friction between the solids, and thus to facilitate the expulsion f the contained air, thereby promoting the density of the concrete.

With reference to the method of determining the mix used in the sts (p. 348 §), a more proper method would be to find the proportion f cement required to make a mortar of maximum density, and then determine the proportion of that mortar requisite for concrete f maximum density. With regard to the water/cement ratio, that

§ *Ibid.*

Mr. Mitchell. was not necessarily the best water/cement/aggregate ratio, the latt being affected by the degree of absorption of the aggregates, and the amount of moisture in and on them. In that connexion referen might be made to the remarks, on p. 333,§ as to the advisability, i making test-cubes, of having the first portion of concrete to be p into the mould somewhat wetter than that which was to follow That conformed to good field-practice, the reason being that give by the Author, namely, " the excess water from the lower stratum concrete will very quickly pass up into the mix thrown in on top it, which will, in turn, reject what it cannot hold." Where th aggregates were exposed to the contingencies of the weather, as wa usually the case, it was obvious that the proportion of water require to make a properly workable concrete would vary very considerabl A heavy thunder-shower might have a disastrous effect on concreting operation. The use of dry concrete was a reversion to a older practice. There was a time when, relying on the results laboratory experiments, engineers had seemed to vie with each othe as to who would use the smallest proportion of water in his concret It needed some serious failures to convince them that results tha might be obtained in a laboratory were not necessarily characterist of good field-practice. One of the drawbacks of dry concrete wa that it very quickly lost its workability, and therefore did not contai the necessary provision for meeting the various delays between th mixer and the place of deposit which were inseparable from ordinar working-practice. It seemed probable that that might have bee the cause of the variability of the strength-results referred to in th Report of the Sub-Committee on Vibrated Concrete (p. 439 ‡). I making comparisons between the relative strengths of dry and w concretes, it was desirable that tests should be made after a long period than 14 days—say at 3 months—and there seemed groun for believing that at the end of the longer period the wet concret would give better relative results.

The Author. The AUTHOR, in reply, pointed out that Mr. Broomfield would doubt be able to obtain data regarding the effect of varying th ratio of mould-size and shape to aggregate-size when the furth reports of the Building Research Station were published. Th Author regretted that he was unable to devote any further time the present moment to that side of the research.

So far no useful data had been obtained in regard to in-situ wor and some considerable time was likely to elapse before a scienti examination of that aspect of the problem would be made. As w

§ *Ibid.*

‡ Journal Inst. C.E., vol. 5 (1936–37), (March, 1937).—ACTING SEC. INST. C.

ointed out in the Paper, the variables present in in-situ work were The Author.
ery considerably greater than those in pre-cast work. Again, most n-situ work involved the placing of a very much larger number of ubic feet of concrete than was ever likely to be handled in precasting, and hence it was unlikely that the whole of the concrete involved would be given the same acceleration, especially when the vibration was to all intents and purposes applied from a point. Dr. Hamlin ound that when using asynchronous vibrators, nodes and antinodes were formed along the shuttering and in the mass of concrete. From he figures supplied it would appear that each vibrator covered an ffective area of approximately 15 square feet. Assuming the wall f the tank to be 9 inches thick, the mass of concrete vibrated would e in the neighbourhood of 1,700 lbs. If the vibrators were of the nduction-motor type running from a 50-cycle supply they would ave a speed of rotation of approximately 3,000 revolutions per ninute, and hence they would generate 3,000 vibrations per minute, r 50 per second.

Then from the formula $P = Ma$,

$$P = 1{,}700 \times \text{amplitude} \times (2\pi f)^2,$$
$$= 1{,}700 \times 4 \times \pi^2 \times 50^2 \times \text{amplitude}.$$

r $$P = 17 \times 10^7 \times \text{amplitude, nearly.}$$

It was clear from that that if the amplitude were of any magnitude reater than about 0·001 inch, the force required would be very onsiderable. On the other hand, if the amplitude were of that mall magnitude at that relatively low frequency the acceleration ue to the action of P would be very much less than that due to ravity. The conclusion was that since experiment had shown that ccelerations equal to or less than gravity were ineffective in producing consolidation, the vibrations were not capable of covering such large area, or that vibration applied at right angles to gravity bore no elation to that applied in the line of action of gravity. So far there was no reason to believe that such a lack of similarity of effect existed. Uniform results should be obtainable in consolidating large volumes f concrete both in pre-cast and in-situ work, provided that he technique adopted suited the particular circumstances of mix, tc. Reinforcement, except in exceptional circumstances, that was to say where the natural frequency of a bar coincided with he applied vibration, should offer no difficulty except as regards per-vibrating."

Since the Author's tests had all been carried out on 4-inch cubes nd there had been little likelihood of marked damping of the ibration-wave occurring, there seemed no point in taking cores rom those cubes. Some rather rough-and-ready tests had been

The Author. taken of vibration-plant installed by the Author for the buildi department of the London Passenger Transport Board. Those tes had shown that the vibrations were transmitted vertically over distance of approximately 14 feet from the point at which vibrati was applied. It was, of course, impossible to distinguish betwe vibration transmitted from the base of the casting vertically, a that transmitted horizontally by friction from the sides of t rigid mould.

Mr. Marks had come to the conclusion that the mix had been lea which was certainly correct judged from the standpoint of comm practice. The effect of vibration was, however, to produce an exce of cement-paste, which came to the surface on all sides of the cub Experiments carried out later on the production of bricks had show that a concrete made of 1 : 4 mix by volume and pressed into t moulds would not give as good a strength as concrete made 1 : 5·5 mix which was vibrated into the mould. The workabili of the two mixes was approximately equal. The strength of t vibrated brick was three times that of the pressed brick. T estimate of the volume ratio was very nearly correct, the slig difference being due to the fact that 94 lbs. per cubic foot was excess of the figure determined for the rapid-hardening ceme used in the research. The true water/cement ratio was not affect by absorption, as both the sand and stone used were specially chos for their non-absorbent qualities, but it was possibly affected separating-out of the excess water from the mix during vibratio although the water carried with it a certain amount of cement a fines. It was found on analysis that the weight-ratio had increas from 1 : 6 to as much as 1 : 7·5. The density of the hand-ramm cubes varied very considerably, but an average figure would as Mr. Marks suggested, from 146 to 147 lbs. per cubic foot.

The discrepancy between the density-curves for 6,000 vibratio per minute was due to plotting and drawing each curve separate and not tracing one from the other ; in any case the slight variati was of no consequence. Only those curves shown in *Figs. 1* and (pp. 320 and 328 §) related to the first series of tests. The cur relating to the second series in *Fig. 6* was so marked. The lack compaction at the lower accelerations and at high frequencies, su as at 12,000 vibrations per minute, was due to lack of kine energy (see Table I, p. 319 §).

There seemed to be no way in which a decrease in the settin time of cement-paste could be effected by vibration, since the acti of vibration was purely mechanical and consisted in compacting t

§ *Ibid.*

lids forming the cement paste, whereas the setting-time of the ment was a function of the surface offered by each particle to emical action. The Author.

While no actual measurement of temperature-conditions had been ken of the concrete during vibration, it was doubtful if the energy sorbed in overcoming friction between particles was sufficient to use any but the most minute change in temperature, and hence e hypothesis of "flash set" being caused on that account had to abandoned.

Mr. Mitchell had pointed out that consolidation could be effected applying the vibration at right angles to the direction in which avity acted. It had not yet been demonstrated that the results oduced in a given mix of concrete by applying the vibration at ght angles to, or in line with, gravitational action were equal or en related. It was safe to suppose that the accelerations given to e particles forming the concrete could possess two components at degrees to each other. That no doubt was what actually did cur, and it was when the force acting vertically tended to zero at the lateral pressure on the walls of the mould was reduced. he concrete, which tended to behave as a liquid, without a liquid's n-compressibility factor, would tend to alter its position, and us each particle which had been slightly compressed would expand ainst its neighbours, causing the "twisting and turning" remarked on by the Author. As a rule the water-content, even in the case very dry mixes, was greatly in excess of that required for proper dration of the cement. There was no doubt in the Author's mind at the failure of engineers to produce satisfactory results with y mixes was due to the fact that plasticity of mix was not obtained placing, and hence proper cohesion between particles never took ace.

Paper No. 5087.[1, 2]

"Flood-Hydrographs."

By Bertram Darell Richards, B.Sc. (Eng.), M. Inst. C.E.

Correspondence.

Mr. J. S. Alford thought that every attempt to rationalize pirical formulas was a step in the right direction. It seemed Mr. Alford.

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[1] Journal Inst. C.E., vol. 5 (1936–37), p. 405. (March, 1937.)

[2] In the Paper, p. 408, line 17, *for* $v = \sqrt{ds}$ *read* $v = c\sqrt{ds}$; p. 425, line , *for* $h^{\frac{3}{2}} \times C_6 h = C_7$ *read* $h^{\frac{3}{2}} + C_6 h = C_7$.

Mr. Alford. desirable to aim at the establishment of a group of expressions relat to each other, governing on the one hand the estimation of run-o from small urban areas, and on the other hand the flood-dischar from a large catchment. Clearly there was a limit to the size of catchment which could be dealt with in that manner. Where t relation i/I (p. 407 §) was non-existent or practically meaningles the problem passed into the realm of probability.

In the report of the Committee appointed by the Ministry Health in 1929 on Rainfall and Run-off, rainfall-intensities in relatio to time were given for use in normal cases for the design of urba storm-water drainage-systems. The practice had grown up designing such works as though the rain started suddenly, fell at th precise rate of intensity laid down in the report, and ended sudden or continued at a greatly reduced rate, although the report was sile on such points. One example of the practice was the use of the area time diagram. The use of that diagram appeared to obscure rath than to clarify the principal point at issue, which was the determin tion of the run-off at a given time and place arising from rain falli under ascertained or assumed conditions.

In order to obtain the required result for a catchment served a main channel with several affluents, it appeared necessary to pl the hydrographs for each affluent or principal affluent. Tho having been superimposed suitably in accordance with defermen a covering graph for the main channel could be obtained who ordinates were the sum of the ordinates of the hydrographs of t affluents. In that way the need of having a record of the who storm, or alternatively the need of making definite assumptio about the commencement, continuance or falling-off, was forced the attention of the computor.

Mr. Binnie. Mr. W. J. E. Binnie, Vice-President, observed that the Auth had considered the question of floods from a theoretical point view, having taken into consideration the intensity and duration rainfall, run-off coefficient, extent and shape of drainage-area, a slope of the ground.

The hydrographs which were shown in *Fig. 5* on p. 17 of t Floods Committee's Interim Report [1] represented normal conditio which obtained in Great Britain for upland drainage-are Assuming those conditions to be correct, the Author had calculat the maximum intensity of run-off for catchment-areas varying extent from 1 to 25,000 acres. The curves plotted from his resu coincided almost exactly with the curves derived from actual flo

§ Page numbers so marked refer to the Paper. (Journal Inst. C.E., vo (1936–37), (March, 1936).)—Acting Sec. Inst. C.E.

[1] Inst. C.E., 1933.

cords as regarded the maximum rate of flood-discharge between those nits of area (*Fig. 3* of the Committee's Report), and the periods taken r the flood to arrive at its maximum intensity, as shown by the drographs in the Committee's Report, agreed fairly closely with s calculations. When, however, the catchment-area exceeded ,000 acres, the Author arrived at a smaller normal rate of flood-scharge by percentages varying from 25 per cent. for a catchment-ea of 75,000 acres to 50 per cent. for 450,000 acres. As the diagrams -relating the maximum rate of flood-discharge with the extent of e catchment-area contained in the Committee's Report represented rmal conditions, exceptional cases would occur where modification as required. Mr. Binnie.

The Paper contained two diagrams (*Figs 4* and *5*, pp. 412 and 413 §) owing how the maximum intensity of flood-discharge and the period quired to reach that intensity would be modified by variations in e maximum rainfall, slope of the ground, shape of the drainage-area, d coefficient of run-off. It was not possible, owing to the scarcity information, to check those diagrams with actual gaugings, but ey were based on logical deductions and should prove of service.

Fig. 7 of the Floods Committee's Report gave curves which showed e extent by which the length of the overflow weir could be reduced ing to the "lag" effect of the reservoir, assuming conditions which re illustrated by the hydrographs. Those curves were arrived at a somewhat tedious process of calculation, and the Author had ven formulas in his Paper by means of which the "lag" effect uld readily be determined for conditions which differed from those ustrated by the hydrographs in the Committee's Report.

Mr. H. J. F. GOURLEY considered that the Author had evolved two eful diagrams (*Figs. 4* and *5*, pp. 412 § and 413), to show to what tent the period of concentration and the maximum flood were ered by variation in the several factors involved : those diagrams uld serve as a guide to engineers having to deal with catchment-eas other than those having the average characteristics assumed by e Floods Committee Report.[1] *Fig. 6* (p. 415 §) showed a close reement with the normal maximum floods defined by the Com-ittee's curve, and the times of rise of the flood as determined by the thor were in fairly close agreement with those shown in *Fig. 5* of e Report. The Report, incidentally, stated that a variation of 50 r cent. above or below the times assumed in drawing the flood-drographs had no material effect on the resultant reservoir-lag. Mr. Gourley.

The Author's observations on the somewhat scattered flood-points r the larger areas (as given in *Fig. 4* of the Report) in relation to his

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§ *Ibid.*

[1] Inst. C.E., 1933.

Mr. Gourley. curve for $R = 4$ in *Fig. 10* (p. 420 §), were fair criticism. T Committee had been at a disadvantage in not having rainfall-recor available for those floods, and if the Author's curve were appro mately correct—and there seemed to be no reason to suggest oth wise—the consequence would be that the Committee's curve f normal maximum floods on the larger areas, if adopted, would gi a larger margin of safety than was intended for such floods.

The method of determining reservoir-lag effect described by t Author was a distinctly useful contribution; it enabled the resu to be obtained with relative ease and rapidity and, even if it ga results differing slightly from the more accurate and much mc tedious step-by-step process, it had to be borne in mind that t prognostication of the peak rate of inflow was bound itself to liable to no less error.

Mr. Lacey. Mr. J. M. LACEY pointed out, with regard to widespread rainfa that a description of widespread rainfall in Southern India h been given in a Paper by Mr. E. W. Storey,[1] whilst a description h also been given [2] of the results of a cyclonic storm which had cross the Coromandel coast near Madras in November, 1903, and of t resultant floods in the Penner river; a description of the floods in t Upper Palar river basin, and of their destructive effect due to t same cyclone of November, 1903, had been given in a further Pap by Mr. E. W. Storey.[3] The following Table showed the floods the Kundair river, a branch of the Penner river,[4] at the Raj anicut (situated near Prodatur) in October, 1889.

The catchment-area of the river above the anicut was 2,5 square miles, the length of the main stream was about 66 miles, a the mean width of the catchment was about 38 miles. The rainf on the catchment-basin from the 14th to the 17th October, 1889 wa

Date: 1889.	14th October.	15th October.	16th October.	17th October.
Station.	Rainfall: inches.			
Nandial . .	1·02	6·57	0·63	0·25
Koilkunta .	1·33	0·30	5·20	1·00
Prodatur . .	0·35	0·45	1·46	0·10

§ *Ibid.*

1 "Extraordinary Floods, Southern India: their Causes and Destructi Effects on Railway Works." Minutes of Proceedings Inst. C.E., vol. cxx (1897–98, Part IV), p. 66.

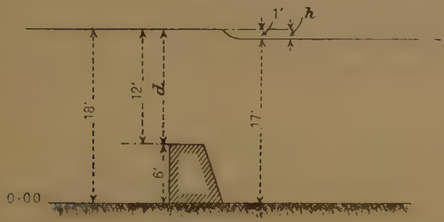
2 J. M. Lacey, "Floods in Southern India." *Ibid*, vol. clxxi (1907–(Part I), p. 360.

3 "Description of an Extraordinary Flood which occurred on the 1 November, 1903, in the Palar river, its Cause, and Destructive Effects." *Ib* vol. cciv (1916–17, Part II), p. 410.

4 *See* Minutes of Proceedings, Inst. C.E., vol. clxxi (1907–08, Part I), *I* *1* (p. 361).

Nandial was at the source of the river, Koilkunta was about 40 Mr. Lacey. iles above the Rajoli anicut, and Prodatur was south of the icut. The rain-gauges were read at 8 a.m. every morning, and the nount gauged denoted the rainfall for the preceding 24 hours; us the rainfall of 6·57 inches on the 15th October at Nandial noted the rainfall at that station between 8 a.m. on the 14th and a.m. on the 15th October, 1889.

Figs. 12.



Formulas:
Clear over fall $Q = 3{\cdot}56\, L d^{\frac{3}{2}}$
Submerged weir $Q = CL\sqrt{2gh}\,(d - \frac{h}{3})$,
neglecting velocity of approach

The following Table showed the water register at the Rajoli icut from the 15th to the 17th October, 1889. The length of the icut was 964 feet and its height above bed was 6 feet. The zero the gauges was the base of the anicut (*Figs. 12*). The figures on that agram represented the readings at the time of the maximum od:

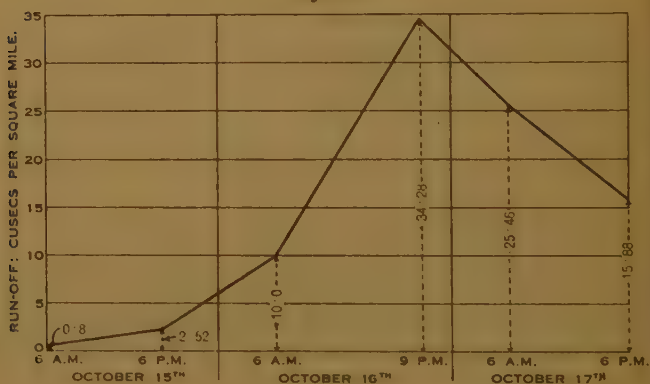
Date: ctober, 1889.	Time.	Gauge reading: feet.		Discharge: cusecs.	Coefficient used: *c*.	Run off: cusecs. per square mile.	Rainfall: inches in 24 hours
		Above Anicut.	Below Anicut.				
15th	6 a.m.	6·70	4·50	2,000	0·666	0·8	0·03
,,	6 p.m.	7·50	4·7	6,305	0·666	2·52	0·094
16th	6 a.m.	9·50	6·50	25,095	0·75	10·00	0·372
,,	9 p.m.	18·00	17·00	85,714	0·95	34·28	1·275
17th	6 a.m.	15·00	14·00	63,600	0·95	25·44	0·946
,,	6 p.m.	11·00	7·00	39,700	0·70	15·88	0·58

It was difficult from the data given above to correlate the rainfall the basin with the discharge at the Rajoli anicut. He would sume that the high rainfall at Nandial of 6·57 inches for the 24 urs preceding 8 a.m. of the 15th October had been concentrated about that hour at Nandial; the first high flood had occurred at a.m. at the anicut, giving a period of concentration of 24 hours,

Mr. Lacey.

or a velocity of flow of about 2·7 miles per hour. If the heavy ra fall of 5·20 inches gauged at Koilkunta on the morning of the 16 were considered to have concentrated at Koilkunta that morni it should have reached the Rajoli anicut in 40 ÷ 2·7, or in abo 15 hours, namely, at 9 p.m. of the 16th, thus resulting in the hi flood at the anicut at that time and date. The high flood of 9 p. of the 16th was bound, however, to have been partly due to the hea rainfall at Nandial gauged on the 15th, and the continuing hi level on the 17th was bound to have been due to some of the ra gauged at Koilkunta on the 16th. *Fig. 13* showed the rise and f of the flood at the Anicut, and it might be observed that genera a rising flood-discharge was convex, and a falling one concave.

Fig. 13.



If the mean rainfall on the basin were considered as having be gauged on the morning of the 16th (that was to say, if 2·43 incl were considered as chiefly contributing to the maximum flood 9 p.m. of that date at the anicut), then the run-off would have be about 50 per cent. of the rainfall, which agreed with the resu obtained in Mr. Lacey's Paper,[1] "Floods in Southern India."

The Author, on p. 407 §, gave a formula $\frac{i}{I} \times f(a)$, in which i denot the average intensity of rainfall, I denoted the maximum intensi of rainfall, and a denoted the area of the catchment. Profess Walsh had given a formula [2] showing the relation between t

[1] Footnote 2, p. 454.
§ *Ibid.*
[2] *Discussion on* "Areas Covered by Intense and Widespread Falls of Rai by J. Glasspoole. Minutes of Proceedings Inst. C.E., vol. 229 (1929–30, Part p. 185.

erage depth of rainfall over a storm-area to the maximum fall Mr. Lacey. ithin that area; had the Author verified that formula?

On p. 408 § the Author assumed t, the time of concentration, to given by $t = \frac{L}{v}$, and that v was constant; he also assumed that $= c\sqrt{ds}$, where $d = it$ multiplied by a coefficient K, that was to y, he assumed that the water accumulated on the catchment and d not begin to run off until the time t had elapsed.

Floods were the result of rain falling on the ground faster than it uld run off; it was possible that as the period of rainfall increased e depth of flow off the ground would increase, but it would rarely ach the value it, which was the state when the rate of fall equalled e rate of run-off.

Mr. G. E. LILLIE observed that for the purpose of his problem, Mr. Lillie. e Author started with two fundamental principles which might accepted without reserve.

Mr. Lillie would state them in the following way:—

The average intensity of rainfall (i) over an area was an inverse function of the size of that area, and (ii) at any moment during a storm was an inverse function of the time the storm had lasted up to that moment.

Then followed two assumptions which needed consideration:—

(i) that for the purpose of the Paper the rainfall was uniformly distributed over the area, and

(ii) that the maximum flood occurred when the whole catchment area was contributing.

In practice the rainfall was never uniformly distributed, and, if e catchment-area were considerable, the storm might well have ased altogether before water from the remote parts arrived and e whole catchment was contributing. Nor was it obvious that en in cases of full contribution a greater rate of discharge might t have been attained before full contribution was established. e question therefore arose whether any great error would result om the Author's assumptions, when used in the somewhat limited ses involved in reservoir-work, for the purpose of preparing formulas r practical application.

The circumstances in which a record discharge might be expected re when a severe storm was centred over the discharge-point (that s to say, over the point where the water entered the reservoir) over the river or main stream feeding the reservoir. Possibly e effect would be even greater if the centre of the storm were over

§ *Ibid.*

Mr. Lillie. the river, and the storm itself were moving down the river to t reservoir at about the same rate as the river was flowing.

In such a case the contribution from the remoter parts of t catchment would be comparatively small, for several reasons, which two were those implied in the Author's fundamental principle (i) the intensity of precipitation of rain on elements of area remo from the centre of the storm was an inverse function of the distan away, and (ii) as it would take time for water from those remo parts to arrive at the discharge-point and the intensity of the stor was subsiding, the rate of precipitation at the centre would ha diminished before that water from the remote parts had begun arrive.

Mr. Lillie thought that that made it clear that the importan of an element of area in contributing to the maximum rate discharge was an inverse factor of its distance from the discharg point. That was the fundamental fact underlying the deductio contained in his Paper on the discharge from catchments,[1] and was expressed in the integral $\int \frac{dw}{y}$ which gave the sum of the valu of the elements of area over the whole catchment. The Author wa however, dealing with only small catchments which might be tak as being within the ambit of a single storm, and hence it was possib that the simplification of the problem attained by the Autho assumption was permissible.

It would be interesting, therefore, to compare the results obtain by the Author's methods and those arrived at by calculations wi Mr. Lillie's formula given in the above-mentioned Paper. Th formula for small catchments was simply $4\Sigma(\theta L)$, θ denoting t included angle and L the radius of various approximate sectors in which the catchment was divided for calculation. Before worki out those figures, however, it was necessary to make one or tw remarks in order that the comparison might be reasonably interprete

In the first place, the Author in *Fig. 10* (p. 420 §) contemplat catchments as large as 700 square miles in area. Those were mu above the size of catchment to which the Author's formulas and tho of the Floods Committee could reasonably be applied, but Mr. Lil noticed that the Table in the Appendix to the Paper did not conte plate catchments of more than 40 square miles.

Secondly, the curve given by the Committee connecting the avera intensity of rainfall over catchment-areas of various sizes with t

[1] "Discharge from Catchment-Areas in India, as affecting the Waterway Bridges." Minutes of Proceedings Inst. C.E., vol. ccxvii (1923–24, Part p. 295.

§ *Ibid.*

eatest spot-intensity, and the similar curve[1] given by Col. Sir Gordon earn, were logarithmic curves, neither having any mathematical igin but being empirically produced from observation. They were obably correct. Both were curves showing the average intensity rainfall in maximum storms over areas of various sizes. On very nall catchments, however (and that was the point to be re- embered), the water could run off as fast as it fell, and as it bviously could not run off faster, those curves were themselves urves of maximum rate of discharge for very small catchments, y 2 square miles in ordinary country or 4 square miles in steep and ountainous catchments. Mr. Lillie.

TABLE I.

Area of catchment:		Rate of discharge by Lillie's formula:		Rate of discharge by Author's formula: Cusecs per 1,000 acres.		Remarks.
housands of acres.	Square miles.	Cusecs.	Cusecs per 1,000 acres.	R = 4.	R = 8.	
2	3·1	5,131	2,565	1,000	2,700	Author's *Fig. 6.*
4	6·2	7,420	1,855	800	2,200	
8	12·4	10,500	1,310	600	1,600	
12	18·6	12,810	1,068	480	1,350	
16	24·8	14,700	920	405	1,170	
20	31·1	16,240	810	370	1,020	
25	39	18,620	745	300	870	
30	46·8	20,300	677	280	800	
50	78	26,600	532	200	600	Author's *Fig. 10.*
100	156	36,750	367	140	430	
150	234	44,800	299	120	360	
250	390	58,100	232	80	285	
350	546	73,500	210	75	250	
450	702	91,000	202	65	220	

Accordingly, Mr. Lillie gave in Tables I and II the figures for the aximum rates of discharge calculated by his own, the Author's, and earn's formulas, the latter being $Q = 640A(4 - \log A)$, in which Q as in cusecs and A was in square miles. The information for very nall areas was, he thought, particularly useful.

In the third place, the maximum rate of discharge was affected by e velocity of flow over the catchment, that was to say by the slope

[1] *Fig. 3* (p. 273), in "The Effect of Shape of Catchment on Flood-Discharge." inutes of Proceedings Inst. C.E., vol. ccxvii (1923–24, Part I).

Mr. Lillie. of the country. Accordingly, the Author assumed an average slo[pe] which he took from the Floods Committee's Report.

Mr. Lillie's formula was for the area of flood-section at the mome[nt] of the maximum rate of discharge, and that flood-section was n[ot] affected by the slope of the country. In order to get the corre[s]ponding figure for the maximum rate of discharge, it was necessa[ry] to multiply by the velocity (which was naturally affected by th[e] slope of the country), and accordingly a reasonable average veloci[ty] had to be assumed, in the same way as the Author took an averag[e] slope. Mr. Lillie had taken V as 7 feet per second.

TABLE II.—CASE OF VERY SMALL CATCHMENTS BY HEARN'S FORMULA.

Area: square miles.	Rate of discharge:		Remarks.
	Cusecs.	Cusecs per 1,000 acres.	
0·25	860	5,380	See the remarks above with regard
0·5	1,500	4,690	Hearn's formula. Except in ste[ep]
1	2,560	4,000	mountainous country it is inapplical[ble]
2	4,230	3,300	above 2 square miles, and above th[at] area it begins to give too large resu[lts] for the discharge, with a quickly i[n]creasing error.
3·1	5,690	2,840	This figure is too large.

Fourthly, in all but the smallest areas, up to say 2 square mile[s], the shape of the catchment made a difference, and he had assumed [a] catchment of breadth about one-half of its length—namely, Type [?] of *Fig. 9* (p. 315 ‡) of his Paper. If the breadth were equal to t[he] length the discharge-rate would be some 10 per cent. greater on [a] 20-square-mile catchment, and 25 per cent. less if the breadth we[re] only one-quarter of the length.

The figures in columns 4 and 6 of Table I should be compared.

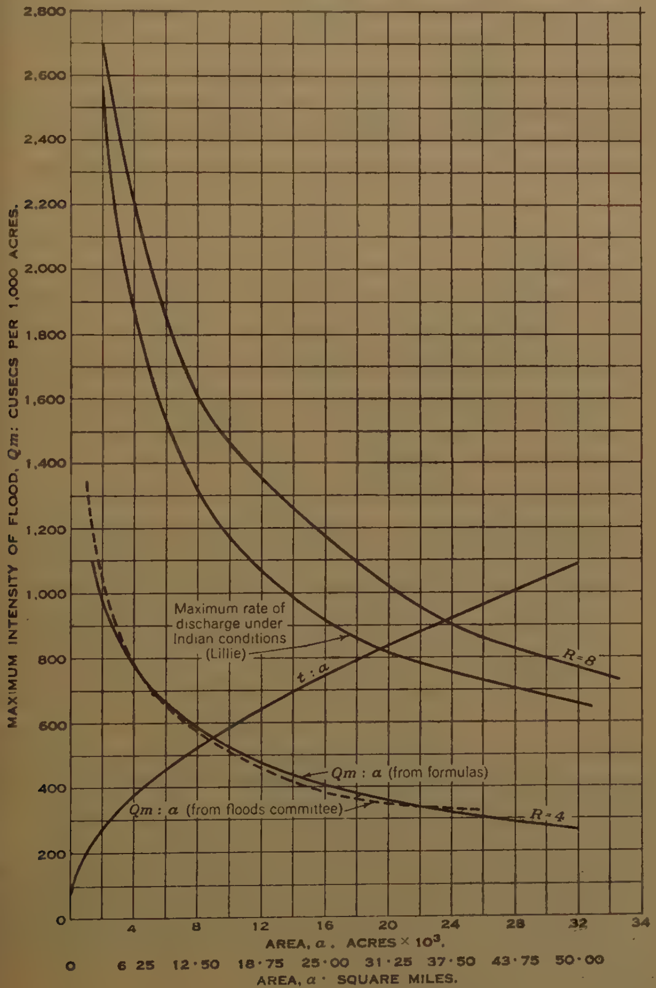
The figures given in column 4 of Table I were plotted, togeth[er] with the Author's *Figs. 6* and *10*, in *Figs. 14* and *15* (p. 462).

From *Figs. 14* and *15* it would be noticed that the curve for Indi[an] conditions (which was well authenticated) lay wholly between t[he] Author's two curves for $R = 4$ and $R = 8$, but nearer to the latt[er]. The Indian curve seemed likely to cross the Author's curve f[or] $R = 8$ for catchments larger than 700 square miles, but Mr. Lil[lie] did not think that the Author's curves were admissible for catchme[nts] of 700 square miles; nevertheless, there was no doubt that it wou[ld]

‡ Minutes of Proceedings Inst. C.E., vol. ccxvii (1923–24, Part 1), p. 315.

ross it somewhere, for it was certain that from large catchments of ome thousands of square miles the maximum rate of discharge in ndia would be much greater than in England. Mr. Lillie.

Fig. 14.



In the Correspondence on Dr. John Glasspoole's Paper [1] he had iven for comparison the figures for the Thames (which had a catch-

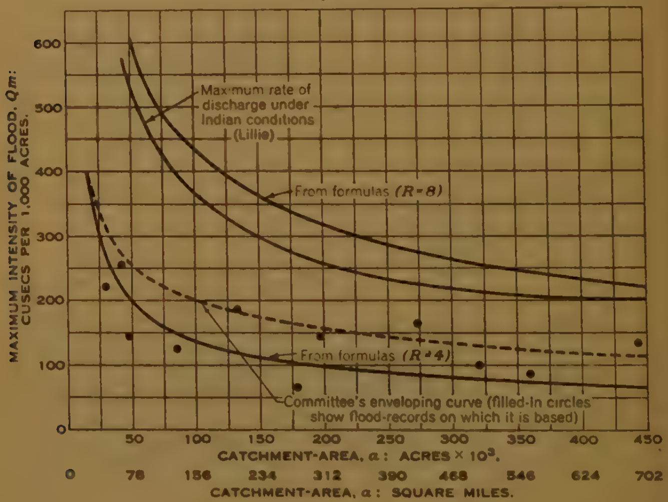
[1] "The Areas Covered by Intense and Widespread Falls of Rain." Minutes f Proceedings Inst. C.E., vol. 229 (1929–30, Part 1), p. 137.

Mr. Lillie.

ment of 5,300 square miles) and the corresponding figures for India rivers of the same area of catchment in districts where the annu rainfall was about the same as in England. The Indian figur were five times as much as those for the Thames. There was r doubt that the maximum rate of discharge from large Indian catch ments was always much greater than the corresponding figures i England, and the reasons were not far to seek.

The action in large and small catchments was, however, quit different, and it did not follow that small catchments should b expected to show greater discharges in India than in England simpl because large catchments did.

Fig. 15.



As stated above in connexion with Hearn's formula, the maximu rate of discharge from a very small catchment was the same as t maximum total rainfall on it. It followed that, in the limit, t maximum possible rate of discharge was the same as the maximu possible spot-intensity of rain, and would occur on the smalle conceivable catchment; from that up to 2 square miles the discharg would be the same as the rainfall, and above that again the run-o would be less than the rainfall.

How did the maximum spot-intensity of rain in tropical countri compare with the same figures in England? He could see no reaso why it should be different; the phenomenon was the same and t weights of air and water were the same all over the world. In Indi 4 inches per hour on one spot had been recorded fairly often, and thos

ates had lasted for an hour or more; probably greater intensities Mr. Lillie. ad happened and had passed unrecorded.

Dr. Glasspoole's Paper quoted many severe cases of extreme spot-ntensity in London, from which Mr. Lillie cited the following. On th May, 1915, and on other dates, 10 inches per hour lasting only minutes, 6 inches per hour lasting 5 minutes, 6 inches per hour asting 10 minutes, 5·5 inches per hour lasting 20 minutes, 4·68 inches er hour lasting 30 minutes and 2·2 inches per hour lasting $1\frac{1}{2}$ hour. t was, he thought, abundantly clear that the spot-intensity could e as great in England as in India, for it might be added that quite s severe falls occurred in the country districts in England as in London, ut had not been so carefully recorded. Similarly, in India the ntense falls were not confined to the wet districts, or indeed to any istricts, as some of the most severe records came from the Sind esert.

It seemed probable enough that the maximum spot-intensity as the same all over the world, although the fall might last longer the tropics. It followed from that that the maximum rate of ischarge from very small catchments might be solved for universal pplication, and that conclusion was supported by the curves in *ig. 14*.

He also believed that the gradient of the intensity-curve from the entre of the storm outward was much the same everywhere; in onsequence, for small catchments up to say 40 square miles, the ecords of discharge would not be greatly different in the tropics. bove that figure the fact that storms lasted longer would make he tropical figures greater and greater in proportion to figures or Great Britain, until, in very large catchments, the disparity as considerable.

That fact was of great importance and ought to be recognized; meant that for small catchments the maximum rates of discharge ight be taken to be the same everywhere, and they were stated in ables I and II. The slope and the shape of the catchment remained s local factors, however, and would affect the rate of flow, but even hat would not affect the maximum rate of discharge in small catch-ents below 2 square miles.

Mr. P. J. Robinson agreed that the factors affecting floods were Mr. Robinson. varied and complex that no precise mathematical computation as possible. There was no need, therefore, for hesitation in dopting approximations that led to simplification in the formulas.

The form of the fundamental expression $I = \dfrac{R}{T+1}$ was somewhat nfortunate, since it led to an equation of the third degree for the valuation of t.

Mr. Robinson. For values of T greater than 1 hour, when $R = 4$, I could be closel[y] represented by

$$I = \frac{2{\cdot}3}{T^{0{\cdot}770}}$$

Similarly, the factor $f(a)$, for areas greater than 2,000 acres, could be represented by

$$f(a) = \frac{1{\cdot}08}{a^{0{\cdot}115}}$$

Equation (1) (p. 408 §) could, therefore, be written

$$i = \frac{2{\cdot}484}{T^{0{\cdot}770}\ a^{0{\cdot}115}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1a)$$

Substituting in the Author's formulas for the period of rise of the flood, and putting $L^2 = Ma$, where

$$M = \frac{n + \dfrac{1}{4n}}{0{\cdot}64},$$

equation (2) became

$$t = 0{\cdot}665\left(\frac{MC}{Ks}\right)^{\frac{1}{2{\cdot}23}} . a^{\frac{1}{4}} \quad . \quad . \quad . \quad . \quad . \quad (2a)$$

With $K = 0{\cdot}6$, $s = 0{\cdot}03$, $n = 1{\cdot}67$, $C = 0{\cdot}014$, $M = 2{\cdot}843$, it resulted in

$$t = 0{\cdot}95a^{\frac{1}{4}}$$

Assuming that K was constant during the whole period of the flood, the total effective precipitation (that was, the amount of rain that ran off as flood-water) was Kit.

Substituting in ($1a$) the value of t from ($2a$), and calling the effective precipitation V, the result expressed in inches of rain was

$$V = 2{\cdot}261\left(\frac{MC}{s}\right)^{0{\cdot}1} . K^{0{\cdot}9} \quad . \quad . \quad . \quad . \quad (3a)$$

or by volume, since 1 inch of rain on 1,000 acres = 3·63 million cubic feet,

$$V = 8{\cdot}207 \times 10^6\left(\frac{MC}{s}\right)^{0{\cdot}1} . K^{0{\cdot}9}$$

With the values for the factors as previously assigned,

$V = 1{\cdot}47$ inch of rain, or $5\frac{1}{3}$ million cubic feet per 1,000 acres.

With regard to the maximum intensity of flood discharge Q_m some confusion appeared to have arisen. The quantity 1,000 K

§ *Ibid.*

n equation (3) (namely, $Q_m = 1{,}000Ki$), was a function of the rain- Mr. Robinson.
all and represented the mean rate of effective precipitation expressed n cubic feet per second per 1,000 acres of catchment. The peak ate of flood-discharge depended on the shape of the hydrograph; nd that in turn depended on the characteristics of the catchment-rea. For certain specific shapes of hydrograph, equation (3*a*) held ood, but as a general expression for Q_m it was incomplete.

The general expression for Q_m was as follows :—

let A_2 denote the whole area of the hydrograph.
,, A_3 denote the area up to the end of time t.

Putting $A_3/A_2 = f_R$, then $1{,}000Kit = A_2 = A_3/f_R$, since the whole rea of the hydrograph was bound to represent the total run-off.

But $A_3 = Q_m f_s t$, where f_s was the ratio of the mean rate of discharge luring the time t to the peak rate of discharge.

Therefore, $1{,}000\ Ki = Q_m \dfrac{f_s}{f_R}$.

Putting $1{,}000\ Ki = Q_R$,

$$\text{then } Q_m = Q_R \frac{f_R}{f_s} \quad \ldots \ldots \quad (4a)$$

If Q_m were equal to Q_R, $\dfrac{f_R}{f_s}$ was bound to have a value of unity. With triangular hydrograph symmetrical about the vertical through the eak, $\dfrac{f_R}{f_s} = 1$, and in general with an unsymmetrical hydrograph, it ras possible in certain cases for $\dfrac{f_R}{f_s}$ to equal unity.

Taking the Author's equations (4) and (5) for the growth of the ood, and assuming, as he had done in *Fig. 3* (p. 411 §), that the atchment was rectangular in shape, (4) might be written

$$Q_1 = Q_m \frac{rb}{Lb} = Q_m \frac{r}{L}$$

hence
$$Q_1{}^2 = Q_m{}^2 \frac{r^2}{L^2} \quad \ldots \ldots \ldots \quad (5a)$$

But, from (5), $\dfrac{r^2}{L^2} = \dfrac{t_1{}^3}{t^3}$. Substituting in (5*a*),

$$Q_1{}^2 = Q_m{}^2 \frac{t_1{}^3}{t^3}, \text{ and } Q_1 = Q_m \frac{t_1{}^{\frac{3}{2}}}{t^{\frac{3}{2}}}$$

§ *Ibid.*

Mr. Robinson. Integrating between the limits $t_1 = 0$ and $t_1 = t$ for the part are of the hydrograph,

$$A_3 = \frac{Q_m}{t^{\frac{3}{2}}} \int_{t_1=0}^{t_1=t} t_1^{\frac{3}{2}} \, . \, dt = \frac{2}{5} Q_m t.$$

As the hydrograph was symmetrical, $f_R = \frac{1}{2}$ and $\frac{f_R}{f_s} = \frac{\frac{1}{2}}{\frac{2}{5}} = \frac{5}{4}$,

whence $Q_m = \frac{5}{4} Q_R$.

As a further example, a trapezoidal hydrograph similar to *Figs. 1* (p. 423 §) could be taken. In that figure the initial storage wa represented by the rectangular area $0AD8$.

If the percentage initial storage had to equal $100 \frac{\frac{1}{2}t_2}{\frac{1}{2}t_2 + t_3}$, it coul be proved that, when $t_3 = t_1$, the ratio

$$\frac{t_2}{t_1} = \frac{\sqrt{(1 + 8\epsilon)} - (1 - 4\epsilon)}{2(1 - \epsilon)}$$

where $\epsilon = \frac{r}{1 - r}$, and $r = \frac{Q_o}{Q_m}$.

When $\frac{Q_o}{Q_m} = \frac{1}{10}$, $\frac{t_2}{t_1} = 0{\cdot}46$, and with that ratio established it coul be shown that $f_R = \frac{1{\cdot}01}{1{\cdot}56}$ and $f_s = \frac{1{\cdot}01}{1{\cdot}46}$; consequently $\frac{f_R}{f_s} = 0{\cdot}936$.

The Floods Committee's hydrograph for an area of 40 squar miles, exclusive of the initial flow, had an $\frac{f_R}{f_s}$ value of $0{\cdot}48$.

Fig. 16 showed the relative values of Q_m for four hydrographs c equal area of the forms dealt with above, the hydrograph conformin to that of the Floods Committee being chain-dotted.

With the values of i and t from (1*a*) and (2*a*),

$$Q_R = \frac{3{,}428}{a^{\frac{1}{4}}} \left(\frac{s}{MC}\right)^{0{\cdot}845} . K^{1{\cdot}845} \quad . \quad . \quad . \quad . \quad (6a)$$

or with $K = 0{\cdot}6$, $s = 0{\cdot}03$, $n = 1{\cdot}67$, $C = 0{\cdot}014$, and $M = 2{\cdot}843$,

$$Q_R = \frac{1{,}564}{a^{\frac{1}{4}}}.$$

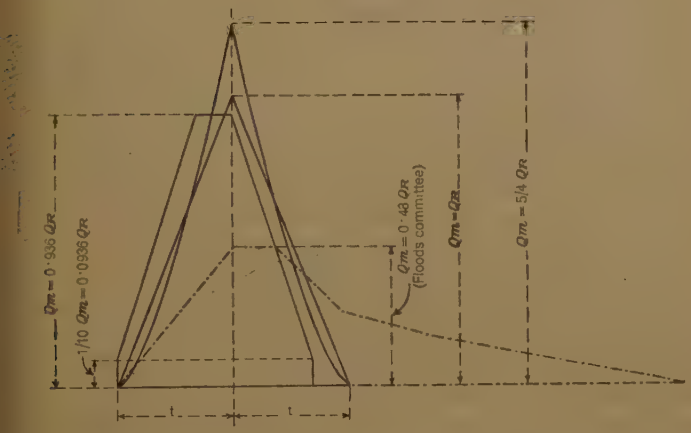
§ *Ibid.*

The close agreement shown in *Fig. 6* (p. 415 §) between the curve marked $Q_m : a$ (from formulas) and the Floods Committee curve was illusory. The former curve was actually a curve of Q_R, whilst the latter was an empirical curve of Q_m based on actual records. **Mr. Robinson.**

The Floods Committee stated in their Interim Report [1] that their simplified flood-hydrographs represented total run-offs (exclusive of initial flow) ranging from 80 to 90 per cent. of 3 inches of rainfall.

The Bransby Williams formula for t, adopted by the Floods Com-

Fig. 16.



mittee, could be written $t_w = 1{\cdot}13a^{0{\cdot}4}$. Accordingly, for $a = 25{\cdot}6$ (40 square miles) the value of Q_R was $\dfrac{2{,}400}{1{\cdot}31 \times 25{\cdot}6^{0{\cdot}4}} = 583$ cusecs, which was nearly double the Author's value, whilst for $a = 1{\cdot}28$ (2 square miles) it was 2,180 cusecs.

The former of those two values of Q_R was identical with the theoretical value given by $K \dfrac{4}{t_w + 1} \cdot f(a)$ with $K =$ unity, and the latter was about 450 cusecs in excess of that value.

It was axiomatic that the area of the hydrograph was bound to represent the run-off. Taking the Author's own formulas for a catchment-area of 26,000 acres with a period of concentration of 4·75 hours, the effective precipitation was 1·425 inch of rain, or a run-off of 5·173 million cubic feet per 1,000 acres. The corresponding hydrograph (*Figs. 11*, p. 423 §), had an area, inclusive of the initial

§ *Ibid.*
[1] Inst. C.E., 1933.

Mr. Robinson. storage, representing about 5·45 million cubic feet. As the rai that produced the initial flood was to be included in the total amoun of rainfall, the area of the hydrograph was in excess of the theoretica run-off. The volume for the triangular hydrograph with no initia flood was not stated in the Paper, but from the examples given i the Appendix it appeared that it had been taken at about 3·8 millio cubic feet. It was thus deficient to the extent of at least 1·3 millio cubic feet, or by about 20 per cent.

The hydrographs shown in *Figs. 7* and *8* (pp. 416 and 417 §) wer similar to *Figs 11*, and had like discrepancies, and the Author' arguments with regard to their shape did not appear to be sound.

In the Floods Committee's Interim Report overflow-weirs wer divided into two classes, A and B. Class A weirs were of a length t discharge the peak rate of inflow with the selected head on the wei whilst Class B weirs were of a length to discharge the peak rate o outflow with the same head allowing for the full cumulative effec of lag. The ratio of Class B lengths to Class A lengths could b denoted by m, and that ratio was equal to the ratio of the peak rat of outflow (or weir-discharge) to the peak rate of inflow.

As he had pointed out in a recent article, [1] the value of m depende entirely on the shape of the flood-hydrograph and the volume o temporary storage above weir-level at the peak of the weir-discharg and could be arrived at without reference to specific values of wei length, coefficient of discharge, or reservoir-area.

The value of Ph, P being the percentage reservoir area, was measure of the amount of temporary storage, and, in his opinio the most satisfactory method of exhibiting the lag-effect was to sho values of m against values of Ph. If a comparison for differin hydrographs, or of differing methods of evaluation, were to be mad the comparison, to be a true one, ought to be made for the same valu of Ph.

The Author's formulas (1) and (3) of Part II of the Paper could b combined and reduced to the following, which was in a for convenient for graphical solution :

$$Z^{\frac{3}{2}} = \frac{435{,}600}{A} \,.\, \tan\theta \,.\, r^{\frac{2}{3}}\,(1 - r) \,.\, Ph$$
$$+ Z\left[\tan\theta(1 - r)\left(1 - \frac{435{,}600}{A} \,.\, Ph\right) + r\right] \quad . \quad . \quad . \quad (7a)$$

where $Z^{\frac{2}{3}} = m$ and $r = \dfrac{Q_0}{Q_m}$.

§ *Ibid.*

[1] "Reservoir Lag," *Engineering*, vol. cxxxvii (1934), p. 422.

With no initial flood and a triangular hydrograph, (7a) reduced to Mr. Robinson.

$$m = 1 - \frac{435{,}600}{A} . Ph \quad . \quad . \quad . \quad . \quad . \quad . \quad (8a)$$

which was the equation of a straight line.

He had pointed out in his article in *Engineering*[1] that if a definite hydrograph applicable to one catchment were taken as a type, and the hydrograph for any other catchment-area were derived from that type (by multiplying all the ordinates of the type by the ratio of the peak rate of inflow for the catchment under consideration to the peak rate of inflow of the type, and all the time-intervals of the type by the inverse of that ratio), then, under those conditions of relative shape and constant area, the value of m corresponding to any given value of Ph became a constant for all catchment-areas; it was, therefore, only necessary to derive m for the type hydrograph.

For that to hold, it had to be assumed that the characteristics of the catchments were similar; granted that that was so, expressions put forward above for t and Q_m showed that

$$\frac{Q_{mc}}{Q_{mT}} = \frac{Q_{Rc}}{Q_{RT}} = \frac{a_T^{\frac{1}{2}}}{a_c^{\frac{1}{2}}},$$

and

$$t_c = t_T \frac{a_c^{\frac{1}{2}}}{a_T^{\frac{1}{2}}} = t_T \frac{Q_{mT}}{Q_{mc}}.$$

Further, the effective precipitation given by (3a) was independent of the area of the catchment, and from that equation it followed that for all catchments having the same characteristics the areas of the hydrographs would be equal.

It would be seen that the Author's equations in the form given in (7a) and (8a) above agreed with the preceding postulate concerning the value of m; indeed, (8a) went much further: it stated that all triangular hydrographs, no matter of what form, would give the same value of m for a given value of Ph, provided that the areas were equal.

The values of m derived from (7a), with $A = 4{,}567{,}000$ cubic feet, tan $\theta = 1{\cdot}246$, and $r = \frac{1}{10}$, as in the example on p. 426 §, were shown in *Fig. 17* (p. 470) by curve (1). That curve cut the horizontal through $m = 1$ at a Ph value of 2·64. Below that value of Ph the Author's approximate equations could not be applied, for it was impossible for m to have a value greater than unity. The curve evaluated by the step-by-step method would give unity value for m at zero value of Ph.

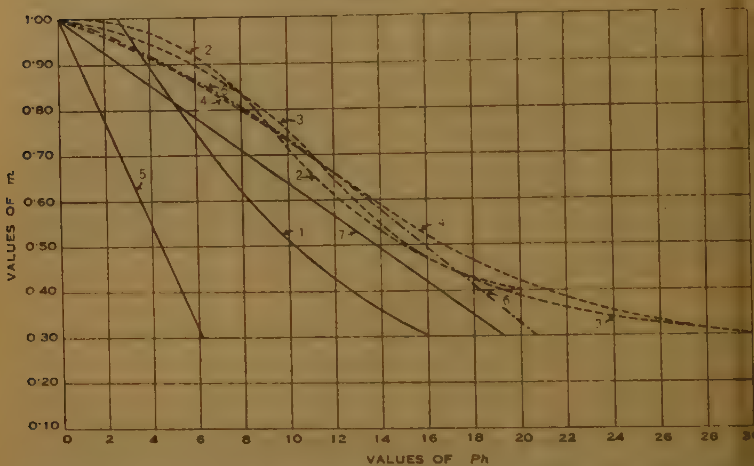
Curve (5) of *Fig. 17* was derived from equation (8a) with $A = 3{,}800{,}000$ cubic feet. Curve (6) was the curve shown in *Fig. 4* of

[1] *Loc. cit.* § *Ibid.*

Mr. Robinson. a Paper that he had contributed to The Institution,[1] replotted agains *Ph* values. That curve had been derived from a triangular hydro graph representing a total run-off of 3·3 inches of rain by the metho of graphical integration advanced in that Paper.[1]

Curve (7) was derived from (8*a*) with *A* equal to 3·3 inches of rain a comparison of the approximate method of the Author with the mor strict method of graphical integration was thus afforded.

Fig. 17.



Curves (2), (3), and (4) were those given in *Fig.* 7 of the Flooc Committee's Interim Report, for permissible heads of 2, 3, and 4 fee replotted against *Ph* values.

It might be remarked that for the specific examples dealt with the Appendix to the Paper, the values of *m* might be derived fro the figures there given, for

$$m = \left(\frac{h}{h_m}\right)^{\frac{3}{2}} = \left(\frac{\text{Col. } 8}{\text{Col. } 11}\right)^{\frac{3}{2}}$$

and $Ph = \text{Col. } (5) \times \text{Col. } (11)$.

The Author. The AUTHOR, in reply, thought that formula (2) on p. 408 § requir some further elucidation, as appeared evident from Mr. Lacey remarks. With the hypotheses taken, if t denoted the time of co centration, $t = \frac{L}{v}$, where v denoted the average velocity of flow the point of concentration, the actual velocity gradually increasi

[1] "The Effect of Reservoir-Area on the Discharge of the Overflow Wei Selected Engineering Paper No. 118, Inst. C.E., 1931.

§ *Ibid.*

The Author.

from 0 to v_{max}. Also $v_{max} = c\sqrt{ds} = c\sqrt{Kits}$, and the average velocity was two-thirds of v_{max}.

$$\text{Hence } t = \frac{L}{\frac{2c}{3}\sqrt{Kits}} \quad \text{whence } t^3 = \frac{9L^2}{4c^2Kis} = \frac{CL^2}{Kis}, \quad \text{where } C = \frac{9}{4c^2}.$$

The factor $\frac{9}{4}$ was not shown in the Paper, but it had been used in the determination of the values of C given on p. 414 §.

Doubts appeared to exist as to whether the formulas of the Paper could be applied to large catchments. The Author had used the records of British floods to check his formulas, as the Flood Committee's Report gave all the coefficients necessary except K, for which he had taken 0·6 as a reasonable value. He had attempted to check the formulas against records of major floods elsewhere, but since such records as he had found did not specify the coefficients, it was impossible to do so satisfactorily. By assuming probable coefficients, a reasonable measure of agreement was found between actual and estimated flood-intensity, but a formula could not be verified on such assumptions without more definite knowledge of the catchment. While, therefore, he could not claim to have proved the formulas as applicable to large catchments, as a personal expression of opinion he thought that they were so.

Mr. Lillie assumed that the Author only intended the formulas to apply to small catchments and cited the Table in the Appendix (pp. 430 and 431 §) as evidence. That Table had, however, been introduced solely to illustrate the method of estimating reservoir lag-effect.

Mr. Lacey had given some very useful data with regard to a big flood on the Kundair river, from which all the coefficients except s could be deduced. They were $K = 0{\cdot}5$, $a = 1{,}600 \times 1{,}000$ acres, $R = 8$, and $L = 66$, giving $n = 1{\cdot}7$, and $C = 0{\cdot}0107$. For s the Author assumed 0·01. Taking $f(a) = 0{\cdot}60$, as it would be approximately if the curve in *Fig. 1* (p. 407 §) applied to India, the maximum flood and period of concentration deduced from the formulas were 35,600 cusecs and 44 hours. The actual record of the flood showed 35,714 cusecs and some time greater than 39 hours.

Whilst agreeing with Mr. Lillie that a storm might cover only part of a large catchment, he would point out that the object of the formulas was to estimate the probable maximum flood-intensity, and that that was obtained with high values of R. Such high values were produced by long-continued heavy rainfall rather than

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§ *Ibid.*

The Author. by intense storms of short duration; long-continued rainfall wa generally widespread and would cover a large catchment-area. In the formulas the average intensity was made a function of both area and time of concentration; that was to say, a theoretical averag intensity was taken of a storm of area equal to that of the catchmen and of duration equal to the time water would take to flow from th most distant part of the catchment to the point of concentration.

It might be argued that, with a high value of R, a small part of th catchment might give a higher flood-intensity than the whole catch ment, and that point he had dealt with in considering the limit o maximum flood-intensity on p. 417 §.

Mr. Lillie had shown curves of maximum flood-intensity fo Indian conditions in comparison with those in *Figs. 6* and *10* (pp. 41 and 420 §). The latter curves, however, represented particula curves for a specified class of British catchments with given coeffi cients. Variation of any one of the coefficients R, K, s, or n, woul modify those curves, so that they were really representative of a lon series of curves. It was difficult to believe that the conditions o catchments over so large a country as India could be standardized t one curve. To do so was in fact to make flood-intensity a functio of the area of the catchment only. That was the weak point o the well-known Dickens formula, and engineers in various parts o India had endeavoured to correct that by varying the constant t suit their particular districts. In fact, they merged all the variable but area into one variable coefficient. The object of the Author formulas was to provide separate coefficients for the various factor affecting flood-intensity.

Mr. Lillie pointed out the fact that the Thames catchment gave flood-intensity of only one-fifth of what might be expected fro Indian catchments of similar area and annual rainfall. The Autho suggested that the explanation might be found in the higher valu of R given by the more prolonged Indian rainfall, and possibly als by a difference in the coefficient K.

With regard to Mr. Robinson's argument that the quantity 1,000 K of formula (3), p. 409 §, did not represent maximum flood-intensit the Author thought that the answer to that lay in the explanatio of the derivation of i.

The evaluation of t from formula (2) presented no difficulty if curve of $\frac{t^3}{t+1}$ against t were plotted; the formulas were very simple t handle, and he doubted whether anything was to be gained by tran posing them into the alternative forms suggested by Mr. Robinson

§ *Ibid.*

CORRESPONDENCE

ON PAPERS PUBLISHED IN APRIL 1937 JOURNAL

Paper No. 5120[1].

"West Middlesex Main Drainage."

By DAVID MOWAT WATSON, B.Sc., M. Inst. C.E.

Correspondence.

Mr. Alford. Mr. J. S. ALFORD asked if the Author would supplement his description by adding to the illustrations a longitudinal section or profile diagram of one of the siphons, preferably the Brent siphon, as the Minutes of Proceedings contained but little information on the subject of the conveyance in inverted siphons of liquid containing solids in suspension.

Mr. Bush. Mr. W. E. BUSH, of Brisbane, remarked on the growth of population shown, and the maximum and average densities to be provided for, as indicating the great problem entailed by the aggregation of population in large cities.

The Paper emphasized also the desirability of the unification of various local interests and the need for wide vision and wise control, unfettered by artificial boundaries, in dealing with the development of all community undertakings.

Whilst it was obviously impossible to give details of all fixtures, he suggested that it would be an advantage if, as a supplement to the diagram shown in *Fig. 18* (p. 503 §), a Table or vertical section could be given showing the entry and draw-off levels of the various tanks, screens and chambers. Would the Author also state the depth of wet sludge that it was proposed to deposit on the drying beds, and the estimated, or, if figures were available, the actual, shrinkage allowed for, and the average period of drying? The different types of backdrops by pipe, ramp, spiral and straight cascades were interesting, but it would appear that, unless there was some special reason for the adoption of other types, the pipe cascade was the best one to instal.

Finally, he would like to learn how the method of natural ventilation was operating in practice. It was assumed that interceptor-

[1] Journal Inst. C.E., vol. 5 (1936–37), p. 463 (April, 1937).

§ Page numbers so marked refer to the Paper.—ACTING SEC. INST. C.E.

Mr. Bush. traps existed on all the house connexions to the sewers draining into the scheme, and that those sewers were more or less naturally ventilated.

Mr. Eddy. Mr. HARRISON P. EDDY, of Boston, Mass., observed that the characteristic which impressed him as being most unusual in the scheme was the rapidity of the growth of the tributary population involving as it did radical increases in plant during the design and construction stages, to provide capacity at the outset of operation which could not have been anticipated at the inception of design. It was interesting to note that the Act of Parliament, under which the works had been constructed, had made provision for the effluen from the sewage-treatment plant " to contain not more than 3 part per 100,000 of suspended matter, and to take up not more than parts per 100,000 of dissolved oxygen, in 5 days at a temperature o 65° F." That an effluent of such high quality could be produced was shown by the record of the North Toronto activated-sludge plant in Toronto, Ontario. For the year 1936 the suspended solids in the final-tank effluent averaged 0·8 parts in 100,000, with a maximum average for 1 week of 2·8 parts in 100,000. The 5-day biochemical oxygen demand of the effluent averaged 1·4 part in 100,000 for the year, with a maximum average for 1 week of 2·4 parts in 100,000.

The method adopted for de-watering the sludge at Perry Oak was of interest because in the United States the trend in sludge drying facilities seemed to be away from drying beds and toward mechanical de-watering, as exemplified by the chemical-precipitation plant at Dearborn, Michigan, the new Calumet activated sludge plant at Chicago and the plain sedimentation-plant for th District of Columbia. Among the reasons for that trend were th slowness with which many sludges gave up entrained water and th consequent large area of drying beds required to de-water a give quantity of sludge, as well as the high moisture-content of th "dried" sludge, particularly the highly colloidal sludge fror chemical precipitation or the activated-sludge process, whethe digested or not.

The loading in lbs. of solids dried and removed per square foot c bed-area in a unit of time was probably the most useful expressio of the work performed by a drying-bed area thus far develope although it had not been employed widely. Messrs. Rawn, Banta an Pomeroy [1] reported that at the activated-sludge plant of the Lo Angeles County Sanitation Districts near Harbor City, Californi sludge dried at the rate of 663 short tons per acre per year on ope

[1] A. M. Rawn, A. P. Banta and Richard Pomeroy, *Civil Engineering*, vo 6 (1936), p. 172.

sand-beds. That was equivalent to an annual average of 0·083 lb. of solids per square foot daily. At that plant, primary sludge and excess activated sludge, seeded with about 20 per cent. by volume of digested sludge, were subjected to multiple-stage digestion before being discharged upon the sand-beds, where the sludge was usually dried to a moisture-content of about 50 per cent. The warm climate of southern California was naturally favourable to the use of open drying-beds. Mr. Eddy.

Data relating to the operation of the glass-covered sludge-drying beds for 1 year at the North Toronto plant were shown in the following Table:

DATA RELATING TO THE OPERATION OF THE SLUDGE-BEDS AT NORTH TORONTO DURING 1935.

Month.	Total solids in sludge applied: per cent.	Volatile matter in solids of sludge applied: per cent.	Total solids in sludge as removed from sludge-beds: per cent.	Yield of sludge-beds in solids per square foot per day: lb.[1]
Jan. . .	7·08	49·0	16·5	0·032
Feb. . .	6·30	50·5	24·1	0·034
Mar. .	6·21	50·1	22·7	0·052
Apr. . .	5·56	51·3	27·7	0·076
May . .	4·84	52·4	34·3	0·089
June .	4·91	52·1	25·8	0·107
July . .	4·68	52·6	24·9	0·108
Aug. .	4·39	52·1	20·3	0·104
Sept. .	4·16	52·1	20·6	0·074
Oct. . .	4·07	52·0	18·3	0·047
Nov. .	3·92	52·3	16·8	0·031
Dec. . .	3·67	54·4	12·8	0·026
Average	4·98	51·7	22·1	0·065

[1] Weighted average, based on number of days sludge was on drying-beds each month.

The yield varied from 0·026 lb. of dry solids per square foot daily in December to 0·108 lb. in July, with a yearly average of 0·065 lb. of dry solids per square foot daily. It was important to note the seasonal variation in yield even where the beds were more or less protected from the weather. Those yield-figures had been derived by a method intended to give a fair measure of the work performed by the drying-area in any month, by taking into account the number of months or parts of months during which sludge had remained on the beds before removal. For example, if it were assumed that sludge applied to the beds in July was removed in September, instead of crediting to the month of September all the sludge removed in that month, it might be more accurate to credit a large portion of the drying of that sludge to the month of August and another portion to July, the respective portions being based on the percentage of the total drying-

Mr. Eddy. period falling in each month. At the North Toronto plant, primar sludge and excess activated sludge were heated and digested i separate sludge-digestion tanks before being discharged upon th drying-beds. The latter were equipped for heating, and artificia heat was utilized whenever it appeared to be of assistance in dryin the sludge.

In contrast with the work performed by the sludge-drying bed in Los Angeles County and North Toronto were the yields of vacuun filters which de-watered undigested sludge at the activated-sludg plants in Milwaukee, Wisconsin, and Pasadena, California. A Milwaukee, the yield of sludge-cake from the Oliver filters had variec from 0·014 to 0·056 lb. of dry solids per square foot per minute equivalent to from 20·2 to 80·7 lbs. per square foot daily. At tha plant the moisture of the sludge applied to the filters had averagec from 98 to 99 per cent. and that of the de-watered sludge had rangec from 82 to 85 per cent. At Pasadena, sludge with an initial moisture content of 99 per cent. had been de-watered on Oliver filters to final moisture-content of from 78 to 80 per cent., with an averag yield of 0·016 lb. of dry solids per square foot per minute, or 23·0 lbs per square foot daily. At both of those plants the de-waterec sludge was dried to a moisture-content of approximately from 3 t 5 per cent. in rotary driers, and was sold as commercial fertilizer.

For final disposal of mechanically-de-watered sludge several nev American plants had turned to incineration. That trend was doubt less due in part to the high moisture-content of the de-waterec sludge. Among those which disposed of sludge by incineration wer the Calumet plant at Chicago and the plant at Dearborn, Michigan in both of which the digestion of sludge was omitted entirely. Th de-watered sludge at Dearborn contained about 67 per cent. moistur when it was fed into the incinerator.[1]

Dr. Engel. Dr. F. V. A. E. Engel was particularly interested in the referenc to the venturi flume and its adaptation as the most suitable measurin device in connexion with the West Middlesex main drainage scheme Everyone would agree with the reasons given by the Author on p 486 § for the preference for a venturi flume for the measurement o fluid flow in gravity sewers. A considerable amount of researc had been done recently with regard to venturi-flume meters, and D Engel [2] had been engaged for some time in investigating the hydrauli

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[1] *Public Works*, vol. 67 (1936), No. 7, p. 42.

§ *Ibid.*

[2] F. V. A. E. Engel, "The Venturi Flume." *The Engineer*, vol. 158 (1934 pp. 104 and 131.

F. Engel, "Venturi-Kanalmesser. Die messtechnischen Eigenschaften i Abhängigkeit von den Strömungsarten." *Archiv für Technisches Messer* vol. 4 (1935), No. 45.

tures as well as the most suitable design of that type of meter. Dr. Engel.
e maximum rate of flow through one of the six flumes shown in
g. 20 (facing p. 477 §) at the outlet from the grit-chambers was
out 100 million gallons per day, and model tests were therefore
e only means of establishing the discharge-coefficient, head-loss
d other important features in the design, since no existing test-
ant could deal with such quantities.

The model was geometrically similar to the prototype and con-
sted of a true copy of the grit-chamber, the venturi flume and the
orm-water overflow-weir. After a careful investigation, the scale
the model had been chosen to be one-tenth of that of the full-
ale plant. Rails were fitted for a point-gauge to take depth-
easurements and numerous tapping-points inside the channel were
nnected to a series of gauge-glasses to determine the pressure-head
a number of sections along the axis of the channel. At each
easuring-section there was a row of tapping-holes in the bottom of
e flume leading into a common duct and thence to a gauge-glass,
d there was a similar arrangement in the side walls.

The venturi flume had to work under free-discharge as well as
der drowned-flow conditions, and for that reason it was necessary
have one tapping-section in the upstream part of the channel as
ell as a tapping-section in the contracted part, or throat, of the
ume-meter. The rate of flow through the flume was given by a
lationship which was based on the Bernoulli theorem. The
ernoulli theorem, however, was only correct under ideal conditions,
d great care had to be taken in using the results obtained from
odel-tests for determining the characteristic features of the proto-
pe. Applying the Bernoulli theorem to the upstream section
uffix 1) and to the section in the throat (suffix 2), it was usual to
ate the relationship in the following way :

$$\frac{p_1}{\rho} + \frac{v_1^2}{2g} = \frac{p_2}{\rho} + \frac{v_2^2}{2g}$$

There were, however, several limitations and deviations from the
ernoulli theorem, which might be dealt with in the four following
ctions :

(1) Dissipation of energy,
(2) Viscosity-influence,
(3) Velocity-distribution, and
(4) Curvature of water-filaments.

Dissipation of energy was not taken into account in the relation-
ip as given above. Across the two tapping-sections upstream and

§ *Ibid.*

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in the throat, the amount of dissipation of energy was assumed to very small, of the order of from 1 to 3 per cent. The dissipation that case was mainly due to friction, as the entrance was very smoo and gradually changing in area, as shown in *Fig. 20* (facing p. 477 thus preventing separation and contraction of the main wat stream in the throat-entrance, which would introduce addition losses. The smooth surface and the large dimensions resulted small frictional losses. To obtain similar conditions in the mod the walls were made out of zinc sheet, and it had to be assumed th no considerable difference in relative roughness existed between t model and the prototype.

No reference was made to any viscosity-influence in the form Bernoulli theorem as given above. On the other hand, viscosi could not be neglected, especially where the water-filaments we converging, which, as previous investigations of Professor A. Gibson had shown, had a stabilizing influence resulting in lamin or streamline-flow conditions existing far beyond the critical veloci That point was of importance when considering the scale of t model. It was evident that the scale had to be such that the Reyno number was high enough to work beyond the range of lamin or quasi-laminar-flow conditions, since results obtained in th ranges could not be applied to the prototype, which worked in a ra of very much higher Reynolds numbers and therefore in a state fully-developed turbulence. It had to be borne in mind that in t case of flume-meters, Froude's Law of Similarity had a predomin influence on the hydraulic characteristics. It was a known fact th having the same Froude number in both the model and the prototy prevented the Reynolds numbers from being the same, and th therefore a considerable discrepancy in the Reynolds Law Similarity might exist. The comparatively low Reynolds numb obtained in a small-scale model had a further influence, as a relativ thicker boundary-layer would exist in the model than in the pro type. That had the effect of decreasing the actual opening of throat, and thus producing a greater differential head for a gi rate of flow than that produced under similar conditions (regard the Froude number) in the prototype for a similar rate of flow.

The difference in the Reynolds number in model and protot would also produce a different velocity-distribution. Two Fre investigators (Coriolis and de Saint Venant) in the last century shown that Bernoulli's theorem as given above was not compl as it neglected the influence of the velocity-distribution. kinetic energy was actually greater than the kinetic energy based

§ *Ibid.*

mean velocity obtained by dividing the rate of flow by the cross- Dr. Engel.
tional area of the water-stream, and therefore it was necessary to
roduce correction-factors on both sides of Bernoulli's equation
the term which referred to the velocity-head. That correction-
tor was greater than unity, but the information at present avail-
le on that subject was very meagre.

Another point which was often neglected in the application of the rnoulli theorem was the influence of the curvature of water-ments, which was particularly important in the case of flow over irs and through venturi flumes. Bernoulli's theorem was strictly ly applicable in cases where no curvature of water-filaments curred; that was to say, where the water-filaments were parallel. cases of great curvature or of convergency or divergency of the ter-filaments, a considerable discrepancy could occur between a essure-head measurement and a depth-measurement. The first erence to that effect which he had been able to trace was given by essrs. A. Koch and M. Carstanjen.* The influence of the curvature water-filaments in the case of venturi flumes had also been brought his attention in a communication by Mr. A. M. R. Montagu, Inst. C.E.

It might be of interest to give some of the test-results which had en obtained in connexion with the investigations on the model the main venturi-flume meters for the West Middlesex drainage heme. Before going into details it was necessary to give some of e dimensions used. It was intended to consider two measuring-ctions, one upstream of the contraction and one situated in the rallel-walled throat and about two-fifths from the entrance. The dth-ratio of the flume-meter was approximately 0·273, for which e critical depth † was about $0{\cdot}674d_1$.

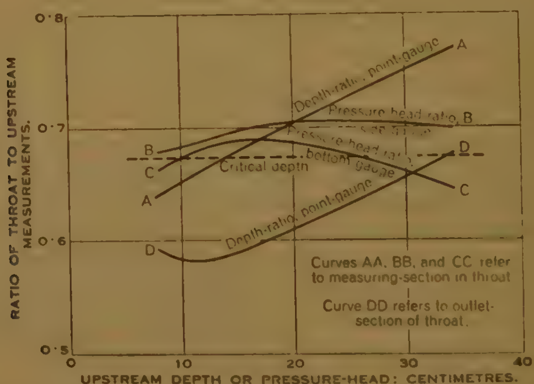
In the upstream section there was practically no difference between e actual depth and the pressure-head measured with the gauge-asses. The discrepancy in the throat-section was, however, very arked. In *Fig. 47* (p. 480) the ratios of depth and pressure-heads the throat section to those of the upstream section were plotted ainst the upstream depth (which, as previously stated, coincided th the upstream pressure-head). The diagram also showed a line dicating the critical depth. It was of interest to note that the ajor parts of the curves AA, BB, and CC were above the line of

* A. Koch and M. Carstanjen, "Von der Bewegung des Wassers und den bei auftretenden Kräften." Berlin, 1926.

† Professor A. H. Jameson had given a relationship from which the critical pth might be easily obtained as a function of the width-ratio: "The Develop-ent of the Venturi Flume." *Water and Water Engineering*, vol. 32 (1930), 105.

Dr. Engel. critical depth, a result which might be rather misleading. T
venturi flume was actually free discharging, that was to say, var
tions in the downstream level had no influence on the upstre
measurement. Therefore somewhere in the throat the critical dep
was bound to have been reached. A further curve (DD) was sho
in *Fig. 47* which referred to the depth-measurement taken at the e
of the parallel-walled throat; that curve, over practically the wh
range investigated, was considerably below the critical depth.
would not be a practical proposition to make the measurement
the throat always at the critical section, as its position varied w
increasing rates of flow, from the entrance of the throat to its outl
The reasons for the selection of the position of the measuring secti
in the throat in that particular case could not be dealt with in t
present discussion.

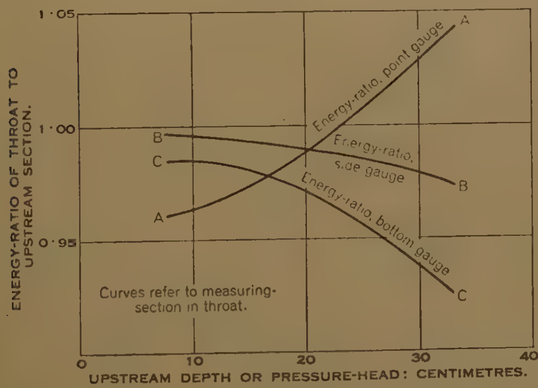
Fig. 47.



The ratio of the total energy in the throat to the total ene
upstream, as given by the Bernoulli theorem, was shown in *Fig.*
The curve AA referring to the depth measurement had been obtai
by adding the velocity-head to the depth of the water-stream
measured by the point-gauge. Similarly, the curves BB and
referring to the gauge-glass measurement giving the pressure-he
had been obtained by adding the velocity-head to the pressu
head measured. He had also made some calculations referring
depth-measurements at the outlet of the parallel-walled section
the throat, but as the curve followed curve AA very closely, it
not included in *Fig. 48*. That diagram was interesting in so far a
clearly indicated that a depth-measurement in the case of a wat
stream with curved filaments was incorrect, since it would indic
a gain in energy shown by values of the energy-ratios above uni

rve BB (*Fig. 48*) seemed to indicate that the "true" arrangement **Dr. Engel.**
s to measure the pressure-head in a case of curved water-filaments
rough a venturi flume which had no hump, by using a float-well
gauge-glasses connected to tapping-points in the side wall of the
roat. For the side gauge, the amount of energy-dissipation across
e convergent inlet did not exceed 2½ per cent., whilst the bottom
uge seemed to indicate a dissipation of energy in the convergent
trance-section of the flume of up to 7½ per cent., which in his opinion
as an excessive value. In the case of the West Middlesex main
ainage plant it was obvious that only a float-well connected to
pping-points in the side wall was a practical proposition. It
ight be mentioned that the Bernoulli theorem as given above
uld be written in an expanded form by including correction-factors

Fig. 48.



d new terms to take into account dissipation of energy, velocity-
stribution, and the curvature of the water-filaments.

In his opinion, model-tests were of the greatest importance in
vestigating hydraulic structures, and especially in investigating
easuring-devices such as weirs and venturi flumes. The numerous
ints dealt with previously might seem to complicate the issue, but
ving the knowledge of all the influencing factors, it was possible,
using correct dimensions and correct methods of measurement, to
ercome the difficulties which arose. It was of interest to note that
e results on the model for the Mogden purification-works were in
cellent agreement with the well-known work of Mr. C. C. Inglis [1]
flume meters.

[1] C. C. Inglis, "Notes on Standing Wave Flumes and Flume Meter Falls." overnment of Bombay, Public Works Dept., Technical Paper No. 15. Bombay, 28.

Dr. Engel. Some confusing statements regarding "critical-depth" meters which class broad-crested weirs and venturi flumes belonged, had b made in previous publications. It should be quite clear, theref that it was neither intended to measure in the section of criti depth nor was it necessary to obtain a coefficient equal to unity, even a constant discharge-coefficient. The measuring-sections to be selected to obtain the most reliable results over a wide meas ing-range. Any variations in the discharge-coefficient could easily taken into account, especially in designs in which the flo movements were transformed into a variation of an electri quantity. The electrical metering system gave a great flexibili especially with regard to incorporating any odd values of hydrau coefficients.

In conclusion, he would like to point out that the express "standing-wave flume" as used in the Paper was slightly confusi In one case it referred (*Fig. 20*, facing p. 477 §) to a venturi flu which might work under free discharging as well as under drown conditions, and in another it definitely referred to a measuring dev which would work always under free-discharge conditions (p. 486 He had shown previously that even in a case of a drowned flu (that was, when the downstream level influenced the upstream c ditions), a "standing wave" might occur, whilst over a wide ra no standing wave would be observed at all. It was therefore s gested that, whatever might be the flow-conditions, the term "v turi flume" should be used, indicating that the type of meter un consideration was working in an open conduit with contracti similar in configuration to the plan of the well-known venturi me for pipelines.

Mr. Osborne. Mr. A. A. Osborne pointed out that on p. 469 § it was stated t Local Authorities were allowed temporarily to discharge exc storm-water into the system. By extending the population-gra (*Fig. 2*, p. 468 §) it appeared that the ultimate population of 2,000, might be reached by about 1954, in which case the expected exc storm-water referred to might require consideration in from 15 20 years' time. If that inferred that certain Local Authorities mi ultimately be faced with the need for disposal of storm-water, complete clearance of existing sewage-works sites as mentioned (10) on p. 467 § might not be possible under those circumstances.

The considerable number of boreholes through water-bear ballast mentioned on p. 470 § raised the question whether any app ciable variation due to lapse of time had been found between water-levels in the working shafts and the original water-lev

§ *Ibid.*

corresponding boreholes, and whether any relation had been Mr. Osborne. erved between such variations and the annual rainfall.

s a scheme of the magnitude of that under discussion involved eful consideration of small details of construction affecting safety of the maintenance-staff, the general use of york-ne steps with a sloping handrail would be preferable to the l-nosed brickwork indicated on the side-entrance steps shown in . 7, Plate 1, which, however, might not be typical. Again, the droom in the short side-entrance gallery shown in *Fig. 5* (p. §) was shown as 6 feet 6 inches but a headroom of only 5 feet s provided in the long gallery in Fig. 7, Plate 1 for the same-size ver; it was hoped that the special circumstances of construction erred to for the long-gallery manhole had not necessitated general ption of a 5-foot headroom instead of 6 feet 6 inches in side rances.

He would be interested in further information with regard to the e of joint used for the concrete pipes referred to at the top of 471 §, and whether ventilation-columns had been used over any the air-inlets mentioned on p. 478 § where the sewer had little ver. How had the brick sewer been eventually safeguarded against detrimental effects of sulphur-laden water on the 2 : 1 cement-rtar joints referred to on p. 482 §?

Mr. H. F. Payne observed that the smallest outside diameter cast- Mr. Payne. n lining used was not given, but from the dimensions of the shield would be approximately 5 feet. Some years ago he had been con-cted with the construction of a sewer in heading in London clay which about 150 feet of 5-foot 6-inch diameter cast-iron lining had en used. The majority of the segments had been made from one uld, and due to some slight error in the positions of the bolt-les it had been found that the lining when erected had developed winding amounting to 90 degrees in the total length. Had any ilar trouble been experienced on the West Middlesex works, and d any precautions been taken to avoid it?

The 12-inch diameter ventilators were described as inlets. Were y means employed to prevent them acting as outlets?

With regard to the formula for the destructive effect of flows in ckdrops, it might be of interest to record that at Dudley, in 1880, re had been constructed a cast-iron drop-pipe with a difference level between the water-surfaces of 86 feet. The maximum charge was about 15 cusecs, so that Qh^2 was 110,940. Up to years ago, when he had left the district, the drop-pipe and bends d never required repair.

§ *Ibid.*

Mr. Payne. Although not stated in the Paper, it was presumed that t[he] sewage-level recorders were of the floating-ball type. If so, he wou[ld] be glad to know if they worked satisfactorily without daily attenti[on]. The Portsmouth Corporation had recently tried one in a sewer a[nd] had found that it was continually stopping owing to deposit on t[he] guide-rods. They had then installed pressure-bulb type instrumen[ts] which were satisfactory.

Mr. Pearse. Mr. Langdon Pearse, of Chicago, observed that the Author h[ad] presented an interesting and concise account of a large undertaki[ng] of particular interest to those engaged in sanitary engineering wo[rk] in the United States because of certain differences in practice fr[om] British methods.

Size of trunk sewers. In the United States a number of cases show[ed] interceptors designed to handle from 350 to 457 U.S. gallons per he[ad] per day on the ultimate population. The ratio of capacity to avera[ge] flow varied approximately from 2 to 4. At Columbus, Ohio, howev[er,] an interceptor to handle from 600 to 2,110 U.S. gallons per he[ad] per day had recently been provided in a special situation.

According to the usual practice in the Sanitary District of Chica[go,] a base-flow was computed from data taking account of the ar[ea,] population, infiltration, and industrial and commercial use, rationalized by a study of the area. To the base-flow a factor w[as] applied for variations in sewage-flow in the seasons and also throu[gh] the 24 hours of the day. Such a factor had ranged from 1·10 [to] 1·60 on an individual sewer, averaging about 1·25 to 1·30. Next [a] factor of 1·20 was applied to cover increments of storm-water. [In] addition, a factor of safety was used to provide for ventilati[on,] unexpected or unusual movements of population, changes [in] industry, and for additional storm water. That had varied in so[me] sewers from 1·375 to 1·5 at the outlet to 2·00 at the upper e[nd]. In general, at the outlet the combined factors amounted to fr[om] 2·1 to 2·25.

The outlet-capacity of the large interceptors varied, ranging fr[om] 0·0234 to 0·0526 cubic foot per second per acre, depending on [the] situation. The present flows ranged from 190 to 330 U.S. gallons [per] head per day. The area tributary to the larger plants of [the] Sanitary District varied from 15,000 to 39,000 acres. In the Sanit[ary] District of Chicago, the trunk interceptors were planned for ab[out] 35 years ahead. The tendency for the water-pumpage to decre[ase] might be accelerated somewhat in the near future, as unive[rsal] metering was approached. The interceptors as planned mi[ght] spill over about fifty-five times a year, including major sto[rms] occurring some six times a year.

Sewers. In the Sanitary District, the intercepting sewers

feet diameter and upwards were built of monolithic reinforced Mr. Pearse.
oncrete in place of approximately 1 : 2·5 : 3·5 mix, with cylinders aving a crushing strength of over 3,000 lbs. per square inch at 3 days. A semi-elliptical cross-section was found to be most ractical for tunnel work. Most of the larger work was built in unnel, under atmospheric pressure. In tunnel, a minimum section f about 4 feet by 5 feet internal dimensions was used. Steel lining as employed in place of timbering whenever work was done under ompressed air. 1 : 3 cement grout was forced in between the asonry and the outside of the excavation at a pressure of 100 lbs. er square inch, through pipes spaced at about 10 feet centres. einforcing steel was used as required.

The detail of connecting large sewers to interceptors was important ecause of the energy to be dissipated. A form of hydraulic jump ad been developed by the Sanitary District, through tests on odels. The connexions required two controls on all large sewers. s the municipal sewers flowed partly submerged, a back-water ate was required. There was also two-float control to limit the dmission to the interceptor, one float operating from the interceptor nd the other from the incoming sewer. That permitted a local orm to discharge into the interceptor so long as its flow-depth llowed.

Sewer-construction. The rate of sewer-construction depended on ne material excavated. In building large sewers in the clay prevant around Chicago, a rate of progress of about 20 feet per 24 hours er heading was commonly made on sewers, even on those with as rge an inside diameter as 17 feet. Labour-saving machinery was eely used, such as electrical haulage, excavation by electric shovels, nd placing concrete by pneumatic conveyors. The use of air-spades ad been very helpful in making tunnelling more economical. laborate precautions were taken to ensure the safety of the orkers, particularly where compressed air was used. Fireproof hafts were required, with a stairway independent of the hoist.

Design of plant. The design of the sewage-treatment works at logden differed from American practice in the inclusion of a separatng weir for storm water, secondary sedimentation-tanks, and stormater tanks. The Sanitary District plants were designed to treat 50 per cent. of the average sewage-flow. A few works in the United tates could handle as high as 200 per cent. of the average flow.

In the Sanitary District works, inclined bar screens were originally sed, with 1-inch openings. After trying $\frac{5}{8}$-inch openings, $\frac{3}{4}$-inch penings were considered the best. The screenings were raked off utomatically on to a travelling belt and delivered to a crusher, in hich they were crushed and returned to the sewage. At the start,

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such screens had been omitted from the Southwest plant. In th newer Sanitary District works, where sludge was to be handled by de watering and incineration, the grit-chambers had been omittec The grit was then deposited in the preliminary settling-basin with detention of from 10 to 35 minutes. In the United States, how ever, grit-chambers mechanically cleaned were generally growin in favour on the larger plants.

In the United States, preliminary settling-tanks ahead of activate sludge were usually short-period tanks. For the Sanitary Distric plants, from 10 to 35 minutes was considered ample. That provide enough fresh solids to mix with the excess activated sludge and t develop optimum conditions for de-watering on vacuum filters. Th sewage dealt with was much lower in suspended solids than was th case in Great Britain. At Milwaukee, grit-chambers and fine screen ($\frac{1}{10}$-inch by 2-inch slots) were used ahead of the aeration-tank The design-basis of major plants in Chicago was :

THE SANITARY DISTRICT OF CHICAGO: DESIGN BASIS OF MAJOR PLANTS.

Plant.	Population: estimated.	Flow in U.S. gallons.		Area served: square miles.	Suspended solids: part per million
		Total.	Per head.		
North Side [1] .	1,100,000	200,000,000	182	115	137
Calumet [1] . .	450,000	136,000,000	300	95	176
West Side [2] .	1,724,000	580,000,000	336	101	121
Southwest Side	1,300,000	400,000,000[3]	308	131	—

[1] Activated sludge.
[2] Imhoff tanks only.
[3] Including waste from Packington.
[4] Average for 1936.

In the United States, the common practice for sludge-remova in settling basins was to use either a rotating mechanism, such as th Dorr or Hardinge, or a straight-line endless chain carrying scraper Both circular and rectangular basins were employed. The tendenc was towards larger basins in order to save construction-costs.

For the sewages in the Sanitary District, aeration-periods of hours had been allowed on a design-basis with an average of fro 0·4 to 0·5 cubic foot of air per U.S. gallon of sewage, with a blowe capacity based on a maximum flow. Operating tests on a larg scale (65 million gallons per day) had shown that a period of 3 hou might suffice. As a rule, in the United States, it was the exceptio to find any provision for sludge-reconditioning; however, in th Calumet works of the Sanitary District, two tanks out of twent

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wo aeration-tanks could be used for re-aeration. The spiral-flow rinciple had been generally adopted because of its simplicity in contruction. The width of the flow-channel at the North Side works vas about 15 feet 8 inches, with two rows of plates. At the Calumet vorks, a width of 33 feet 6 inches had been used, with two rows of lates, but initially some of the aeration-tanks were being operated vith one row of plates. In an experimental unit, there were indicaions that a 68-foot width was possible, with two rows of plates long each side wall. The Southwest plant had a channel-width f about 33 feet, with two rows of plates. Diffuser-plates, 12 inches quare, were the standard. As coarser plates than previously were ıow specified, carborundum or alundum plates, 1 inch thick, were ısed. Formerly silica-sand plates, 1½ inch thick (Filtros), were also vailable in the finer grades of porosity. Those plates were usually et in precast-concrete containers. Cast-iron containers had been bandoned years ago because of rust-conditions. Aluminium ontainers had been tried in a few plants.

At ½ cubic foot of air per U.S. gallon of sewage, the Calumet lant used 3 cubic feet of free air per plate (12 inches square) per ninute, with two rows of plates. Using one row of plates, 6 cubic eet of air were required per plate. The Southwest design was ractically the same. The air was all filtered through oil-dipped leaners before compression.

The regulation of the air to each individual diffuser-plate was not ractised in the United States. In the Sanitary District designs, rom thirty to seventy-two plates were controlled by a common lobe-valve, but in the more recent designs each row was separately alved, from thirty to thirty-six being on one valve. By a system f testing and inspection every plate was rated and marked. In plants he size of those in the Sanitary District, plates closely alike within ery narrow rating-limits were set in a given tank. The water-depth f the aeration-tanks was nominally 15 feet over the plates.

Return-sludge was allowed for on an average of 20 per cent. of the verage flow. At the North Side and Calumet works, a maximum eturn was provided of 40 per cent. of the average, or 20 per cent. f the maximum. At the Southwest works, the maximum was 30 er cent. of the maximum sewage-flow. Whilst there might be a alue in diluting the raw sewage with effluent on the stronger ewages in Great Britain, there appeared to be little need for that ractice in handling the more dilute sewages found in the United States. The difficulties noted in continued drought were those onnected with the behaviour of the sludge as the volatile content ose. Bulking might then be likely to occur. In the North Side and Calumet works, return-sludge pumps were installed. In the South-

Mr. Pearse. west works, air-lifts on individual tank-units were provided, thu giving a localized control. The use of cross-baffles in the aeration channels seemed unnecessary in a design with four channels of total length of 1,600 feet. Out of one hundred and thirty-fiv plants in the United States, only one, of small size (that at Hager town, Maryland), had used a cross-baffle.

In the northern part of the United States, the custom had prevaile of housing over the operating-gallery, both to protect the operato during cold or stormy weather when regulating valves, collectin samples and reading meters, and also to make the controls readil visible. In the milder British winter climate, the omission of th housing was undoubtedly a saving in cost, but might interfere wit the exacting control required in such a plant.

Final settling-tanks in the United States were usually of a typ cleaned with a mechanical scraper similar to the primary tank. Steep hopper-bottoms were found only in very small plants. In th design, care was taken to avoid eddy-currents. An average rat based on mixed liquor, of 1,200 U.S. gallons per square foot of tan area per day was provided for, with a maximum of 1,950 U.S gallons. At the North Side works tanks 77 feet square were use In the Calumet plant tanks 91 feet square were used, with centr feed and peripheral effluent-weirs. At the Southwest Works, tank 126 feet in diameter were used. For a variant on the sweeping devic at Milwaukee a multiple-inlet type of revolving arm ("Tow-Bro" was used, in place of squeegees. The type used at Mogden neces sitated a larger number of units, fifty-two in all, and much deepe excavation than required by the United States practice of large shallow tanks, mechanically cleaned. Operating results indicate the value of larger units and fewer mechanisms to maintain. Wit mechanical cleaning, the Mogden bottom slope might be adequat At the Des Plaines River works of the Sanitary District, however, slope of 2 to 1 was not sufficient on pyramidal-bottomed tanks, th flatter mechanically-cleaned basins proving better.

Storm-water tanks were very rare in the United States an Canada. To Mr. Pearse's knowledge, there was only one plant s equipped in each country (namely Columbus, Ohio, and Nort Toronto, Ontario). It was possible that in the future, where strean flow conditions warranted, the use of storm-water tanks migh become more common.

The operation of the Mogden project would be watched with muc interest to ascertain whether the space allowed per head for sludg digestion proved adequate. The allowance of 1·3 cubic foot primar and 3·3 cubic feet secondary, or 4·6 cubic feet total, did not seem adequate for a mixed activated sludge, if the results on mixed sludg

n the Sanitary District were any criterion. The difficulty in digesting ctivated sludge was to secure a dense sludge. The moisture-content ad ranged from 94 to 96 per cent. with either heat-controlled r Imhoff-tank digestion. Mr. Pearse.

For the Sanitary District, 5 cubic feet per head had proved nadequate for digesting mixed fresh and activated sludge. As a esult, the programme had been developed for the Calumet and South-vest works of eliminating all digestion, taking fresh sludge with the xcess activated, coagulating with ferric chloride and de-watering n vacuum filters, then mixing the wet cake with pre-dried material o produce a water-content of about 50 per cent., drying in a flash lryer, in superheated vapour, and incinerating the sludge by blowing nto a furnace. The heat from the incineration served for drying. All the vapours passed through a temperature of 1,200° F., and all dours were eliminated. That offered a most compact and satis-actory method of sludge-disposal, leaving an inert mass of cinders. The plant was estimated to be more economical than digestion-plant or the Sanitary District conditions. If desired, material dried to 10 er cent. could be withdrawn from the drying circuit and sold for ertilizer. At such times, other fuel was required to replace the ludge withdrawn.

The study of the possible size of sludge-drying beds for the West Side and Southwest Side plants, aggregating over 100 acres of drying rea, had convinced Mr. Pearse that on large projects some form of nechanical stripper was desirable, on special beds, loading direct to tandard-gauge self-dumping gondola cars. For the Sanitary District, a project had been worked out, of which 27 acres had been uilt with the beds 80 feet wide, and units ranging from 800 to ,300 feet long. Two types of stripping machines had been devised, unning on concrete walls, self-propelled and electrically controlled. With each machine, a crew of five men could strip from 1,000 to ,500 cubic yards of air-dried sludge in 8 hours, loaded in 30-cubic-ard dump-cars on a standard-gauge railway. The use of narrow-auge track on sludge-drying beds was decreasing in general practice, xcept in small plants, and small caterpillar-tractors carrying addle-back hoppers were proving popular. In United States ractice, more sand and less gravel was used than at Mogden. In he Sanitary District, at the West Side, the beds were made with 3 inches each (from the bottom upwards) of from ⅞-inch to 1½-inch ravel, of No. 4 sieve to ⅞-inch, and of No. 8 to No. 4 sieve, topped y 5 inches of sand. The subgrade was level. The underdrains in eneral were 18 feet 6 inches centre to centre, surrounded by the oarse stone.

From the design standpoint on large works, operating experience

Mr. Pearse. was very helpful. With the three large plants built in order—th North Side, Calumet and Southwest—the Sanitary District sta had had especial opportunity to observe the behaviour of the plant an its details. Even such matters as paint and non-corrosive meta had been studied over a term of years. As a result, the designs wer developed to produce a low first cost and also low maintenanc costs. The tendency was towards larger units and more compac designs. Flexibility was also very desirable. In the development c the de-watering and incineration procedure, before building th 40-ton-per-day Calumet equipment, over $350,000 was spent i experiments and in two test-plants each of 20-ton-per-day capacity of a temporary character.

The operating results and costs of the Mogden plant would b awaited with much interest, as it was the largest British plant of it type. Mr. Pearse had been concerned with the design of the thre largest plants of the kind in the world—namely, the North Side an Southwest plants of the Sanitary District, and the Wards Islan Plant of New York City—and in each the problems had been some what different. Throughout, however, the question of sludge disposal had been important. The provision of additional area at distant site in the West Middlesex scheme was wise, because of th possibility that both additional sludge-digestion volume and drying bed areas might be required.

He hoped that the Author would be able at a later date to presen a further Paper dealing with the details of design.

Mr. Thrupp. Mr. E. C. Thrupp, of Vancouver, had been the author of the schem for the extension of the London water-supply by storage-reservoir in the Kennet and Enborne valleys to maintain a larger summe flow in the Thames between Reading and the London water-intake One of the objects of that scheme had been to minimize the effec of summer pollutions above the intakes, including those mentioned i the Paper.

One of the most active promoters of the Isleworth half-tide-wei project had been his uncle, the late Mr. C. J. Thrupp, who had bee for many years Chairman of the Twickenham Local Board. Th writer had pointed out that there might be opposition argument put up by some engineers who held that the maximum tidal flo to and fro was necessary to keep the navigable channel below i order. Partly on account of economy, and partly on account c those extreme arguments, the scheme had been framed for a half-tid weir. He had disagreed with those extreme views, and held that th loss of tidal volume would be quite insignificant. The Autho mentioned that considerable erosion had been going on for 20 years a Isleworth Ait, just as it had at Twickenham in previous years. I

oth cases it was due to the momentum of the ebb tide (acting in the Mr. Thrupp. ellmouthed lower reaches which had been improved by dredging elow Greenwich) letting the river down to a lower level and giving steeper surface-grade at low water than the natural ebb stream ad before. The erosion at Isleworth proved conclusively that the eduction of tidal volume had done no harm whatever to the lower iver, and so it would be quite in order to raise the Isleworth weir to igh-water level, and still further to improve the river above. That ed to the question of the desirability of constructing at least two veirs lower down the Thames. In his carefully-considered opinion t would be one of the finest possible improvements for London, and e believed that it would be done.

Whilst admiring the remarkable efficiency displayed in the design nd rapid construction of the West Middlesex scheme, he could ot regard it as the last word in the problem. It was an emergency cheme, well handled as such. The next 30 years would require evised ideas to approach finality. New supplies of water would ave to be brought into the Thames watershed, and sewage-effluents night have to be carried right away to sea, possibly to the south oast, unless they could be purified by further aeration, and by 3 r more weeks' storage in 20-foot-deep reservoirs in successive lternating stages to reach a quality admissible to water-supply ilter-beds. It would be of interest if the Author could state how far he works had succeeded in complying with the specified requirements f purity, and whether the remaining suspended and dissolved natter was of a character liable to give further trouble from bacterial ctivity when exposed to other contamination in the river. Another ossible emergency might arise if the people in England were to ollow the customs of the city residents of the United States and Canada, and insist on having a water-supply of over 100 gallons per ead per day.

Mr. W. P. Warlow observed that the economic diameter of the Mr. Warlow. primary digestion tanks was stated (p. 529 §) to be about 70 feet. Did that apply equally to the stirred and to the gasholder tanks? It did not, apparently, apply to the secondary tanks, which were 100 feet in diameter. Were the walls of the tanks designed to resist the full internal pressure of the sludge? The Author had rightly pointed out that partly-digested sludge was more easily pumped than either raw sludge or digested sludge. In view of that fact, it was presumed that the pipes provided for drawing off separated liquor from the primary tanks were not intended for use under normal conditions.

§ *Ibid.*

Mr. Warlow. The arrangements made for heating the sludge in the primar tanks ensured very effective use of waste heat. The use of a hea exchanger appeared to be contemplated during hot weather fo cooling the circulating water. Could that not be effected equall well by cutting out one or more of the boilers? In any event, wa the Author satisfied that the digestion-process would be impaire if the sludge in the tanks became heated above the "optimum temperature of about 80° F.?

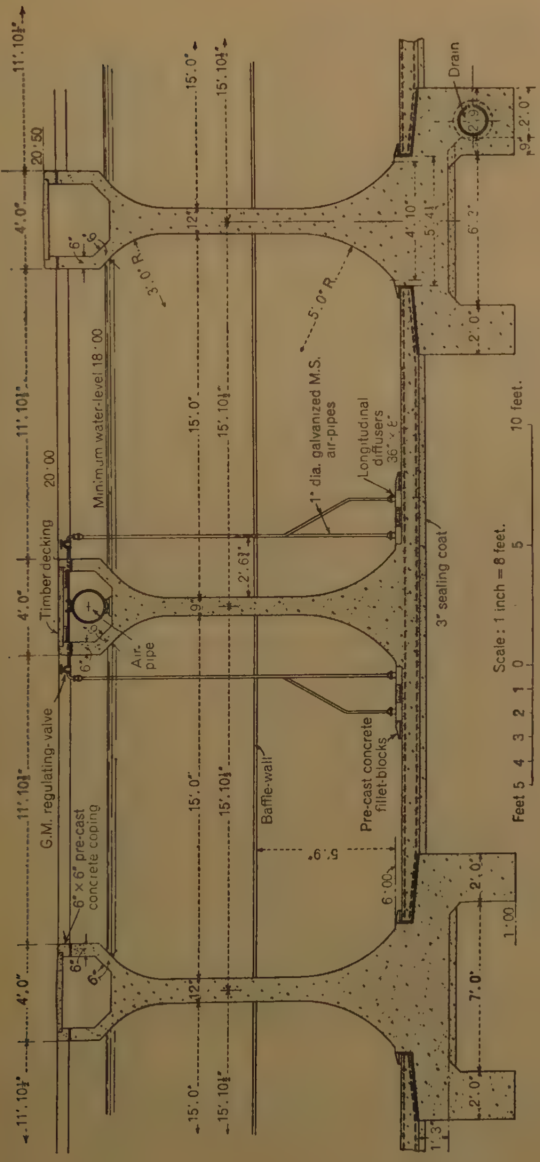
The estimated gas-production of 0·625 cubic foot per head population per day seemed highly conservative, and a figure of cubic foot per day might perhaps have been anticipated from th performance of similar installations elsewhere. Had any precautio been taken to avoid loss of gas by leakage through the concret roofs? Mr. Warlow was aware of two instances in which sligh leakage had occurred, owing apparently to diffusion through con crete.

The value of the Paper to those concerned in the operation sewage-works was greatly enhanced by Mr. Townend's observation in the course of the Discussion. In view of the growing tendency t utilize sludge-gas for power-production, Mr. Townend's statement t the effect that the value, at 2*d*. per therm, of the gas produced a Mogden would be sufficient to defray the whole of the annual cost sludge-disposal was of general interest. That statement wa however, difficult to reconcile with information given in the Pape and perhaps the Author would give further details. The cost the sludge-main and the works at Perry Oaks was stated to b £232,800. If the cost of the primary tanks at Mogden and the land Perry Oaks were added to that figure the total could not be fa short of £400,000. The value of the gas would be about £20,000 p year, which would hardly meet the debt charges (apart from th Government grant), whilst rates, taxes and operation would amou to a further substantial sum, bringing the total annual cos apparently, to considerably more than £20,000 per year.

Mr. Wilkinson. Mr. G. W. Wilkinson considered that as the prime consideratio on most sewage-works was that of purification, some details of th aeration-tanks at Mogden, where the greater part of the process purification was performed, might be of interest. *Fig. 49* was typical cross-section through two channels of the "spiral-flow units, and *Fig. 50* (p. 494) was a cross-section of a "longitudin ridge-and-furrow" unit. The form of the channels was exactly th same except for the arrangement of the diffuser-plates. In bo cases, the form of the cross-section of the channel was considered be such as would give the maximum efficiency of circulation. T use of thin walls of reinforced concrete between the channels h

Mr. Wilkinson.

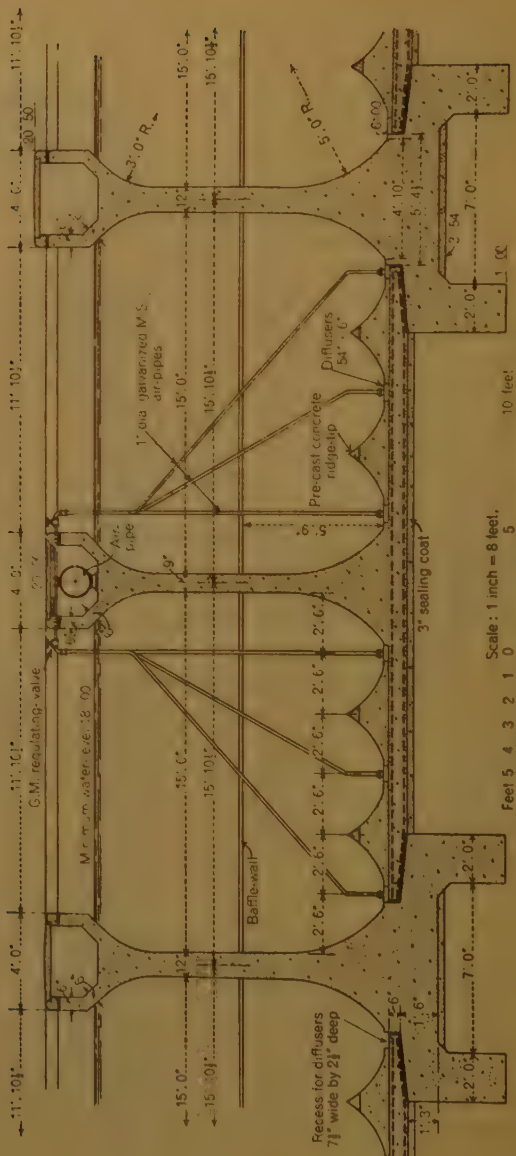
Fig. 49.



"SPIRAL-FLOW" AERATION-TANK.

Mr. Wilkinson.

Fig. 50.



"LONGITUDINAL RIDGE-AND-FURROW" AERATION-TANK.

sulted in appreciable economies in the area of land required and Mr. Wilkinson. the cost of construction.

It was possible to return effluent to any of the aeration-units, and at had been found a great advantage. When, for instance, it was cessary to empty a unit for inspection or other purpose, the inlet-nstock was closed and effluent pumped back, which displaced e mixed liquor, the latter being merely pushed forward to the al separating-tanks. In that way the activated sludge from that it was kept in circulation in the plant. When the tank was nptied, the air-supply could be shut off without any possibility of idge being deposited on the diffusers.

The amount and character of surplus activated sludge produced as a matter of some importance in the operation of activated-idge plants. At Mogden, the quantity produced was about 1 r cent. of the dry-weather flow with a water-content of 98·8 per nt. which could be reduced, by 1 hour's settlement, to 97·6 per nt. The low figure of surplus-sludge production might be accounted r by the efficient preliminary settlement of the sewage before it ached the aeration-tanks, and also by the fact that purification was en taken to the nitrification stage, under which conditions matter oduced by flocculation in the early stage of the aeration-process as subjected to some degree of aerobic digestion.

An inspection of the map of the district surrounding the Mogden rification-works would emphasize the revolutionary changes that d come about in the practice of sewage-disposal. Not many ars ago it had been a *sine qua non* of sewage-disposal practice that e works should be erected as far as possible away from urban velopment, both on the grounds of æsthetics and also with the ject of keeping the smell away from the towns. The site selected r the Mogden works, however, was the natural outfall to the river hames for the greater part of the drainage district, and was for at reason all the more desirable. By means of the activated-idge treatment, the purification of sewage was possible with mplete freedom from smell- and fly-nuisance. No other method uld have been applied with such great advantage to the peculiar quirements of the West Middlesex Drainage Scheme. Indeed, might be said that the feasibility of the scheme was due to that ct.

The application of the activated-sludge process to the Mogden orks represented the culmination in Great Britain of the progress ade within the last 25 years, commencing with the pioneer work such men as Dr. Gilbert J. Fowler, who had initiated the original perimental work leading to the discovery of the process, and of Mr. A. Coombs and the late Mr. Walter Jones, who had been concerned

Mr. Wilkinson. as engineers in translating the laboratory-experiments of t chemists into practical possibilities in full-scale plants.

Second only to the problem of producing a satisfactory efflue was that of sludge-disposal. That, also, had to be accomplish without nuisance, and it was in that respect that the applicati of the practice of sludge-digestion to the requirements of the scher was noteworthy, quite apart from the economies accruing from t utilization of the resultant gas for the power-requirements of t works.

As was well known, the process of sludge-digestion had been evolv at the works of the Birmingham Tame and Rea District Draina Board, and had subsequently been developed by Mr. J. D. Watso Past-President Inst. C.E. To his work in that field especially, th adoption of the process to such advantage and on such a large sca at Mogden was a fitting climax.

The Author. The AUTHOR, in reply, observed that the details of the invert siphon under the river Brent, asked for by Mr. Alford, were show in *Figs. 51.*

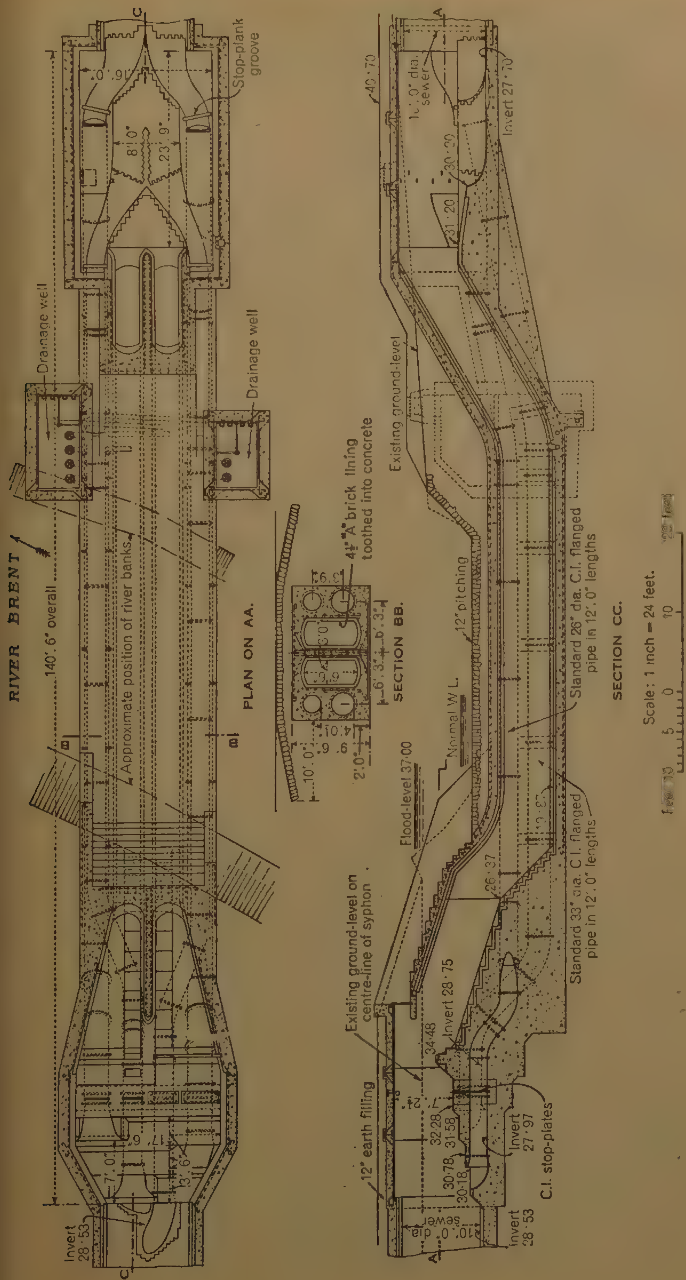
The walls of the sludge-drying beds were 2 feet above mediu level (p. 551 §) and the maximum depth of sludge on the beds cou not therefore exceed that figure, but the other data requested b Mr. Bush were not yet available.

Conditions governing the design of backdrops were very vari and led to the adoption of a number of types. Both Mr. Bush an Mr. Payne referred to the advantage of cast-iron drop-pipes. Com plete destruction of the kinetic energy of the falling sewage of th branch sewer appeared to be more desirable in some cases than i others, and as in so many places the headings and shafts necessa for construction had been available, the opportunity had been take to install some more efficient form of backdrop than the ordina type of cast-iron pipe backdrop.

Ventilation of all sewers, and more particularly large sewers, ha always been a vexed question ; that it could not entirely be divorce from public sentiment and imagination was proved over and ov again by complaints which arose about smell from new sewers n yet brought into operation. The West Middlesex sewers had bee no exception to that form of complaint, and in some cases complain had been received since they had come into operation. In th vicinity of backdrops such complaints had no doubt been justifie but in the main, the principle had been adhered to that the mo the vents the better the ventilation of the sewer, and the less th intensity of the smell at any one point. It was interesting in th

§ *Ibid.*

The Author.



River Brent Siphon.

The Author. connexion to read the Correspondence on a previous Paper [1] whe
Sir George Humphreys showed that the experience in London h
been that public opinion had influenced the decision to close ma
open vents, but that the wisdom of doing so had been doubtf
No means had been taken to prevent ventilation-holes in sew
from acting as both inlets and outlets.

The standard of purity required by the Act, and referred to
the late Mr. Harrison P. Eddy, would be recognized as the norm
standard of the Royal Commission on Sewage Disposal as reco
mended by the Commission in relation to discharge into non-tid
waters—certainly not for discharge into tidal reaches of the Than
such as that at Isleworth. It was not in all respects that Americ
and English practice in sewage-purification and disposal cou
readily be compared, but Mr. Eddy touched on the capacity
open-air sludge-drying beds, and there probably contrasts could
drawn, although differences in climate made comparison under t
same conditions very difficult. In a few years' time, when t
operation of the Perry Oaks works had been stabilized over a reaso
ably long period, the data available could be compared with t
results obtained at other large works in Great Britain and in Americ

Dr. Engel's observations formed a valuable contribution to t
published information on the design of venturi flumes. Incidental
they afforded yet another proof of the whole-hearted co-operation
those manufacturers who were from time to time approached duri
design-stages and invited to help in the solution of difficulties arisi
out of the unusual circumstances of the work, or in the overcomi
of constructional troubles.

As the volumes of storm-water accepted into the County Counci
sewers were generally no less than were the volumes discharg
by the Local Authorities to their now-disused sewage-works, t
operation of the scheme caused no change. All Authorities had h
in the past to make some provision for disposal of storm-wat
other than the volumes discharged into foul sewers. Those pr
visions were unaltered, except that six times the dry-weather fl
of sewage (and, as explained on p. 469,§ even greater volume
were now discharged to the county's sewers, and therefore t
point raised by Mr. Osborne that old sewage-works sites might n
be cleared but might be used for storm-water did not arise in practic
The sites were rapidly being changed for use for other purpose
and in fact many had already been cleared and altered.

No records had been kept to co-relate the water-levels in tri

[1] G. W. Humphreys, "The Main Drainage System of London." Minutes
Proceedings Inst. C.E., vol. cciv (1916-17, Part II), p. 103.

§ *Ibid.*

es and in working shafts. Generally speaking, the only oppor- The Author.
ity available for noticing rest-level of ground-water on the site
a working shaft was when the shaft was being sunk, and not
ays then, since the water was kept down by pumping, and the
und-water level, as such, was lost.

Figs. 7, Plate 1 (following p. 618 §), was typical of side-entrance
nholes in so far as brickwork steps were concerned. Those were
nd to be quite satisfactory in conjunction with handrails and
ety-chains. The headroom of 5 feet shown in Fig. 7, Plate 1,
s a special case, 6 feet 6 inches being the standard dimension.

All concrete pipes had ogee joints and were made with Portland-
nent mortar, that applying also to the comparison mentioned at
top of p. 471 § and referred to by Mr. Osborne. In only a very
cases were ventilation-columns used over air-inlets, and they
re necessitated by location of buildings very near to the sewer.
allowness of the sewer did not influence the question. The case
action of sulphur-laden water on cement-mortar joints of a sewer
d been very carefully observed for a long period before the sewer
s brought into use, and plans had been made for grouting behind
brickwork barrel of the sewer with aluminous cement, but, as
Paper omitted to state, it had been found that when the action
d been going on for some months it stopped completely.

As all bolt-holes in the circumferential joints of cast-iron segment
vers were slotted, it was an easy matter to keep the segments on
even keel, and therefore no winding occurred as Mr. Payne
ted had happened in another case.

The sewage-level recorders referred to by Mr. Payne were assumed
have been all of the guide-rod type, but in West Middlesex all
ats were installed either in chambers or in cast-iron pipes, and
re free to rise and fall as they could not be if they were made to
le on guide-rods submerged in sewage.

All engineers interested in the subject were bound to feel indebted
Mr. Langdon Pearse, of Chicago, for his contribution emphasizing
ferences between American practice and the West Middlesex work.
. Pearse had given much of his time and taken a great deal of
uble to contribute valuable comparative data calculated to help
y engineer designing new works and wishing to weigh up the main
ferences between American and British practice. It should be
nembered, however, in studying the notes, firstly that the con-
nption of water in the United States was very much greater than
Great Britain, resulting in larger volumes of weaker sewage, and
ondly that in Great Britain all volumes of sewage and storm-water

§ *Ibid.*

The Author. up to three times the dry-weather flow were purified to the sa standard.

The cost of cross-baffles in the aeration-channels was trifling, a structurally their existence was useful. Furthermore, they ma convenient gangways.

An answer to Mr. Thrupp's question about compliance with t specified requirements would be found on pp. 580 and 581 § in t remarks made by Mr. Townend. The results given there show that the standard of the effluent had been consistently higher th the Act required, and since Mr. Townend's remarks were ma there had been a continuation of work of the same standard. T effluent discharged was non-putrefactive and the growth of fish-l in the Thames locally was attributable to the improvement in co ditions in the effluent. Live fish were now to be found in the effluer conduits.

It would be incorrect to suggest that the economic diameter both the stirred and the gasholder sludge-tanks at Mogden was t same, but practical conditions dictated that the twelve tanks shou be of the same size. Beyond stating, therefore, in reply to Mr. Warlo that 70 feet was regarded as the economic size at Mogden, and 100 fe at Perry Oaks for a different class of tank built under differe conditions at different depths, it was impossible to do justice that question without going into detail not attempted in the Pape The walls of the tanks were designed to resist the full intern pressure. Digesting sludge, with or without its supernatant wat was usually easier to pump than raw or fully-digested sludge, owi no doubt to gas-content, and therefore as much supernatant wat as possible was in fact drawn off in the primary tanks.

The temperature of the exhaust-gases had always to be reduc before discharge, and the consequent temperature of the water us for cooling them, being in a closed circuit, had to be similarly reduc The heat-exchangers and the sludge afforded alternative means effecting that control. In practice it had been found that the slud temperature might profitably be kept well above 80° F.

A gas-yield of 0·625 cubic foot per head of population had be the minimum assumed. More had been anticipated, and provis had been made accordingly. Double roofs were provided to stirred sludge tanks, but no provision was made to prevent diffus of gas through the concrete.

In referring to Mr. Townend's remarks on the value of gas as co pared with the annual value of the works and plant required to duce the gas, Mr. Warlow raised an arithmetical question.

§ *Ibid.*

thor thought, however, that Mr. Townend's point, dealing solely it did with the economics of that method of sludge-disposal, was pendent entirely on the value of the gas, which Mr. Townend for e sake of his argument put at 2*d*. per therm; if Mr. Warlow's thmetical contention should be correct, it would be only right st to assume an economic, and therefore considerably higher, lue for the gas. As the Author understood Mr. Townend's argunt, he sought to stress the great value of the gas given off, and ereby implied that conditions no doubt existed where the gas uld be harnessed economically. The Author.

Paper No. 5103.

"Welded Joints in Pressure-Vessels."[1]

By Stanley Fabes Dorey, D.Sc., M. Inst. C.E.

Correspondence.

Mr. L. C. Dipper noted that in Germany, Switzerland and France ie range of 22-26-28 tons per square inch ultimate tensile strength as preferred for steels used in the class of welded work under conderation, whereas in Great Britain the lower range usually specified as 26–30 tons per square inch, which the Author considered quite iitable. Was it to be inferred from that that Great Britain had ad greater success in welding the higher-quality material, and if so, as that due to a better technique or to the use of a better class of ectrode? Dr. J. Orr and Mr. W. Heigh, in a Paper before the istitute of Welding,[2] gave the impression that the German prefernce for butt-welds rather than fillet-welds was due in some measure the lower quality of welding in that country. There would appear be some connexion between that fact and the use of lower-quality aterial. Mr. Dipper.

Regarding the presence of the martensitic zone in the case of ustenitic welds, it would be interesting to hear how the hardness of at material was determined, bearing in mind the extreme narrow-

[1] Journal Inst. C.E., vol. 5 (1936–37), p. 621 (April, 1937).

[2] "Impressions of Welding in Germany." *The Welding Industry*, vol. v 937), p. 251.

Mr. Dipper. ness of the band (0·04 millimetre). If the material were true marte site he would expect a higher Brinell number, but in ordinary te might not the result be affected by a "cushioning" produced the softer material beneath the martensite? If the numeral we correct, might that not indicate that the zone was partly troostite sorbite?

Dr. Faltus. Dr.-Ing. FRANK FALTUS, of the Skoda Works, Plzeň, observed th it was very satisfactory that the Author was absolutely in favour welding, particularly of fusion-welding of boilers. His views we almost identical with the views taken in Czechoslovakia so far as t efficiency of welded joints, the necessity of testing them, and t sufficiency of stress-relieving instead of full normalizing were co cerned, the latter being specified in some cases.

In connexion with the procedure-control and testing he would gi some observations based on the studies of his firm in high-pressu boiler-welding, and on some recent jobs successfully carried out their Works.[1] Great importance was attributed in their Works the testing of electrodes, which should, in their opinion, underg just the same tests as specified for and applied to the parent-meta His firm, therefore, asked their electrode-suppliers to submit to the separate test-certificates for every electrode-section of each cast, an particularly of all-weld-metal tensile- and impact-test specimen They specified not only lower but also upper tensile limits to b closely adhered to (for example, from 42 to 50 kilograms per squa millimetre) because they wanted the weld to suit the parent-met as much as possible, and did not want it to form "hard spots" the welded job. Hard steel, even if its elongation and impact te figures were satisfactory, tended towards formation of cracks in th weld during welding. They further specified a higher elongatio (26 per cent. on a gauge-length of 4 diameters) for electrode-tes than was usually specified for the inspection of the finished job. O the other hand, they considered that that test was unnecessary routine-tests.

They fully agreed with the Author with regard to bend-tests. A the first glance the specimen (*c*) (p. 649§) seemed to comply fully wit the requirement of equal testing-conditions for plates of differe thicknesses. But that was not so. A weld joining together thic plates was in its top part far broader than a weld between thin plate and so it might easily happen that just the most important part

[1] There have been welded satisfactorily large high-pressure vessels with wa thicknesses of up to 76 millimetres, and even sections up to 200 millimetres thic —F. F.

§ Page numbers so marked refer to the Paper. (Journal Inst. C.E., vol. (1936–37) (April, 1937).)—SEC. INST. C.E.

e joint, namely the transition from weld to parent-metal, could me to a place, during testing, where it would be less strained (being most unaffected by the deformation). With reference to what had en said above, the free bend-test gave more exact results. His m were well aware of the fact that it was difficult to measure the ongation of the outer fibres of a bent specimen, but a very impor- nt factor was contained in the prescription of 30 per cent. ongation. Electrodes, the tensile strength of which lay not too uch above that of the parent-metal, gave an elongation of more than per cent. without cracking of the weld when bend-tested through en 180 degrees, but a representative weld-elongation could not be tained with a weld having a high tensile strength, even when nd-tested through 180 degrees, because it was the parent-metal at contributed most of the necessary deformation. It might even ppen that the fracture occurred outside the weld. A bend-test ving the specified minimum elongation therefore prevented the e of too hard electrodes which would form "hard spots" in the ructure. Dr. Faltus.

His firm were also in agreement with the Author's assertion with gard to the value of impact-tests, but they would like to mention at, upon their suggestion, the Czechoslovak regulations for the sting of electrodes for steel structures had been modified in the llowing way: "Impact tests are to be carried out by means of x specimens, but the average of five best results is conclusive." hat provided a possibility of excluding a false result occurring cidentally.

They did not attribute great significance to the density-test, it was difficult to imagine a weld complying with all specified quirements, even those of X-rays examination, and failing only the density-test. Their opinion was that the number of tests ould be restricted as much as possible, only the most important es being applied, and they would therefore recommend leaving it the micro-specimen tests.

They were very grateful to the Author for his open appreciation the importance of X-ray examination, and also for his emphasizing e necessity of an exact interpretation of the radiographs. They nsidered it most important to place suitable measuring means ch as a penetrameter on each radiograph. It was their opinion at the best advantages drawn from X-ray examination applied the training of welders and to the development of new welding- ethods. They did not hesitate to declare X-ray examination dispensable for the purpose of those kinds of tests.

Mr. H. B. Fergusson pointed out that the Author had given no dication as to the extra cost of manufacturing pressure-vessels to Mr. Fergusson.

Mr. Fergusson. the requirements of Lloyd's Class I or of the A.S.M.E. Code, and t average designing engineer was still somewhat vague as to wheth it would pay to try to obtain as nearly as possible a perfect weld joint by the insistence on the adoption of one of those codes, with the extra cost of X-raying, mechanical tests, and macro- and micr examination, or whether it would be cheaper to adopt thicker plat and a lower joint-efficiency, and to do away with those rigid spe fications. He was able to assist in throwing some light on th problem, and he found that the extra cost in manufacture of co structing Class I pressure-vessels to one of those codes would invol an additional cost of from 9 to 27 per cent. on the total cost of t job, depending on whether there was a considerable weight internal fixed fittings to the vessel which did not have to be X-raye such as a vessel filled with tubes, or whether the vessel consist merely of a shell with dished and flanged ends. Even taking t high figure of 27 per cent. extra cost on the total value of t job, it would appear that the higher joint-efficiency which t designer could use should off-set the extra cost of X-raying th welds and complying with the mechanical, macro- and micro-rules the codes.

It had naturally to be assumed that the works where the vess were made to Class I Lloyd's requirements had had sufficient expe ience in that class of work to enable them to carry out the work wit out excessive welding-costs, as the removal of weld-metal in order clear the exograph of any sign of lack of penetration or slag-inclusio was a very costly matter, apart altogether from the danger of ruini the plate adjacent to such faults by excess welding setting additional stresses in the plate. In the event of having to ch out a weld to remove a fault, it should be obligatory for the manufa turer first to take a stereo-exograph to locate the depth of the fau and to see whether it should be attacked from the outside or from t inside of the vessel. With a little practice the depth could be asce tained to within $\frac{1}{16}$ inch, and such a procedure obviated, even in t worst cases, cutting through the weld to more than half the depth the plate. The weld-metal could then be put back without danger cracking. Actual experience went to show that welders who h spent a few months on welding 1- to $1\frac{1}{2}$-inch thick plate subject X-ray examination became so expert that, whereas when th commenced they showed some type of fault in every 3 feet or 4 fe of welding, they could easily do 20 feet of welding without a sign any lack of penetration, slag, or flaws of any kind ; when some fla was found on one of the exographs, it generally only consisted from $\frac{1}{4}$ inch to $\frac{1}{2}$ inch lack of penetration on one bead of weldi only. In his opinion the silicon-content in the steel plate should

pt as low as possible, as otherwise there might be a tendency for e weld-metal to crack, especially with thick plate. Mr. Fergusson.

With regard to the preference for single-U over double-U welds, hile it was admitted that the single-U weld was the only practicable e where automatic welding was done, it would appear that on thick ate the double-U was decidedly better if the weld were done by nd, as the very great stresses set up in thick plate by the weld-etal were considerably reduced and proportioned over the whole ickness of the plate, both inside and outside, and were not con-ntrated on the outside of the vessel. The double-U photo-macro-aph (*Figs. 6*, facing p. 655 §) showed how necessary it was to build the welding on the outside of the plate, and to chip and grind it f. The photo-macrograph mentioned showed fine-grain structure the middle and columnar crystallization on the outside, due to ck of annealing by superimposed weld-metal, and it was Mr. ergusson's opinion that bend-tests would not give the same results that columnar crystallization existed on the outsides of the weld.

The Author mentioned the possibility of a crack, if in a certain sition, not being detected on an ordinary exograph picture taken erpendicular to the shell. As the danger in a pressure-vessel of a nall crack growing with time, especially if breathing of the vessel ok place, was of such great importance, it would be interesting to ave his opinion on altering the present method of taking X-ray ctures, and on adopting the same system as Mr. Fergusson did when king stereo-pictures; that was, giving two exposures on the same m at two-thirds the normal time of a single perpendicular exposure. he resulting stereo film was very clear, and gave immediately the stance-in of any intrusions, and it would be difficult, if not impos-ble, for any crack to escape attention. There was practically no xtra cost involved except the few minutes extra of the operator's me in altering the angle at which the rays penetrated through the eld.

Dr. HERBERT HARRIS was in agreement with much that the Author ad expressed, and only in details—sometimes insignificant in impor-nce—did he find anything to criticize. On p. 625 § the Author ferred to the fact that in plate-assembly he found it unusual to rovide any shrinkage-gap. That was not a practice followed by r. Harris. It had to be borne in mind that the first layer of weld-etal was applied to relatively cold and frequently heavy plates; the eld-metal was severely quenched, and a high unit-stress could be duced in that first layer. The metal was therefore not in the best ndition to adjust itself to severe stresses. Again, it was not until the Dr. Harris.

§ *Ibid.*

Dr. Harris. publication of a Paper by Mr. H. F. Hall[1] that an adequate explana- tion of high-temperature cracks in weld-metal had become availabl Those experiments had shown that few steels had appreciabl ductility in the temperature-range between the freezing-point an 1,300° C. Over a range of approximately 200° C. all steels wou therefore seem to be very susceptible to cracking. In weldin therefore, care had to be taken to minimize the stresses imposed o weld-metal. The company with which Dr. Harris was associated ha tackled that problem concurrently with that of distortion-contro and accumulating experience was showing that by allowing a sma shrinkage-gap and minimizing bending at the root of the weld hig temperature cracks were infrequent; when they did occur they we not of a serious character, in that they were readily removed befo the weld was completed. In connexion with the recommendatio that plates should be stress-relieved after bending, but befo welding, he had had such a proposal examined in practice, and as consequence he was of the definite opinion that no advantage wa derived thereby.

The Author referred at some length to the question of plate-materia and "pending a definite lead" (as he put it), he gave on p. 637 § chemical and physical analysis of plate-material which Dr. Harr hoped was not intended to serve as a basis for a steel-plate specifica- tion. Manganese-concentrations up to and including 0·7 per cen presented no difficulty whatsoever in welding, and it was quit unnecessary to restrict the sulphur and phosphorus contents furthe than to a maximum of 0·05 per cent. each, to do so would be t increase the plate-costs without deriving any advantage. As t the physical properties, he had adequate reason for believing that th steel-plate manufacturers would have considerable difficulty i supplying plates, of the considerable thicknesses being utilized a present, which could be said to approach at all closely the ductility figures cited by the Author.

Dr. Harris had discussed the question of fatigue-characteristic in a publication issued by his company. It was there reported tha under actual tests to destruction the welded seams of complete boiler-drums were able to withstand fatigue-conditions better tha those parts of the drums which had been accepted as perfectl satisfactory for many years, and further that recent tests of certai full-section welded joints by Professor B. P. Haigh had invariabl shown fracture in the plate-material remote from the weld.

§ *Ibid.*

[1] "Strength and Ductility of Cast Steel during Cooling from Liquid Sta in Sand Moulds." Second Report (1936), Steel Castings Research Committe Iron and Steel Institute, p. 65.

He was very pleased to learn that as a result of experience the only Dr. Harris. thoritative specification in Great Britain was to increase the joint-ciency of a Class I weld from 82 to 90 per cent. for design-calcula-ns. Such concrete evidence of the increasing reliance that could placed in welded joints was reassuring, and he ventured to suggest at with still further experience the time would come when a still gher joint-efficiency would be allowable for a Class I weld.

As the Author had pointed out, the Izod test was one about which inion varied, but it was not irrelevant to point out that if the test re used as an indication of the life or safety of any pressure-vessel, ich was surely the primary concern of a specification, then the plications of the test-figures should be the same, independent whether the steel was weld-metal, rolled plate or forged steel. If e Izod test had proved useless in determining the life or safety of essure-vessels made entirely from rolled plates or forged steels, ere could be no reason to suppose that the test was to take on a ddenly-realized importance because weld-metal was involved. the same time, there could be no doubt that the impact-test d very little in common with the fatigue-tests as the Author emed to imply on p. 652 §; that was particularly true of weld-etal.

In connexion with the point of view that was often expressed that low Izod figure indicated that an incipient crack would easily and pidly extend, it would appear from his firm's experience of pres-re-vessels that such a statement required amplification. Certainly e converse (namely, that a high Izod value indicated that an incipient ack would be propagated with difficulty) was not true. The basis r that statement was that it was not uncommon to find a weld-etal that had considerable porosity characterized by a high impact-lue, and, as Mr. Lucas had shown, such heterogeneities facilitated e propagation of a crack by acting as "stepping stones."

With reference to *Fig. 5* (facing p. 654 §), he suggested that the termediate microstructure was not of the actual fusion-zone, but as of that portion of the plate-material which had just begun to affected by the welding procedure; the actual fusion-zone would about $\frac{1}{16}$ or $\frac{1}{8}$ inch nearer the weld.

Mr. Henry Headland, of Arapuni, N.Z., observed that from the Mr. Headland. int of view of hydraulic machinery in water-power development, e Paper was of considerable interest because of the present-day ndency to substitute fabricated structures for complicated castings d to replace riveted joints by welding in such items as pipe-lines, eeching-pipes, spiral casings, and draught-tubes, which as a rule

§ *Ibid.*

Mr. Headland. were not so simple in shape as ordinary boiler-drums. Welded l and air-vessels for governors were further applications which ca within the category of pressure-vessels.

Apart from the essential feature of watertight joints, wh were ensured by good penetration and freedom from porosity welded spiral casing offered many important advantages over corresponding riveted structure. Effective butt-welding enab the correct logarithmic shape to be obtained, and the gradua diminishing sections, by elimination of the laps, resulted in a smoo internal surface with a minimum of resistance and disturbance the flow forming the intake-vortex. Butt-welding eliminated changes of thickness and concentration of stress at rivet-holes circumferential seams, and the resultant symmetry and simplic decreased the secondary stresses. In addition, the design w considerably simplified, producing a reduction in weight of up 20 per cent. and the elimination of marking-out and drilling in t workshops. On the other hand, the setting-up time in the fi was probably a little increased, and extra care had to be taken avoid undue distortion, a feature which, however, was not altogetl absent in a riveted spiral casing. Welding had the advantage th the continual noise of riveting and caulking was absent.

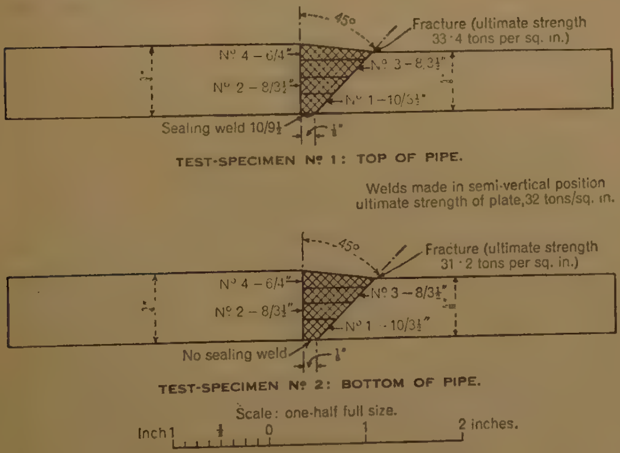
Access for riveting and caulking to many parts of spiral casi was difficult, particularly at the junction of the plates and the st ring. In some recent tests on a large casing for a vertical mach it had been found very difficult indeed to get a structure of th type absolutely watertight without resorting to a certain amou of welding, which had to be carried out with a good deal of ca in order to avoid the imposed loads being taken by the welds inste of the rivets. In vertical machines, it was important to avoid leaka which would impose full static pressure on the surrounding concr during operation, and would also involve difficulties when the finish structure was concreted in place under full pressure. The intr duction of the butt-welded seam permitted the joint to have simi mechanical properties and capacity for deformation to those of t plates, with the additional advantage that the surfaces liable corrosion were always accessible, whilst welds exposed to corrosi could always be made with an electrode specially suited to th conditions.

Metallurgical defects and slag-inclusions acted as points of stre concentration, and when located at or near the outside tension fib they had a considerable effect on the strength of a butt-weld tra mitting bending stresses. The importance of cleaning each r and following the proper procedure for eliminating the crater form at the beginning and end of a run could not be over-emphasiz

enerally speaking, spiral casings were erected in the field, and were too large to be stress-relieved except by mechanical means such as ressing with a roughing tool and by the application of a suitable ydraulic pressure-test. Mr. Headland.

Many of the remarks made above in reference to spiral casings ere also common to pipe-lines. In the case of lap-welds, it was a omparatively simple matter to test the joint by specially shaping he plates at the circumferential seam to provide a space for the ntroduction of pressure-water, whilst with those having butt-welds rrangements could usually be made to test the pipe-line in sections. n pipe-lines the superior strength against impact forces of welded

Figs. 12.



over riveted joints was important when considering the stresses set up by water-hammer. For that class of work butt-welds were preferable for circumferential joints, and were essential for longitudinal seams, in that they were less liable to partial penetration than the lap-weld, which might result in a cavity being left in the corner between the legs of the fillet as well as undercutting. Butt-welds were also an advantage when preparing for sandblasting and painting.

The use of spiral seams in long cylinders under pressure was not mentioned in the Paper, and in the case of pipe-lines it was a problem in economics to decide whether spiral seams, which permitted a 20 per cent. reduction of plate-thickness with the same factor of safety, were justified when the increased costs of rolling and welding were considered. There were several pipe-lines in Europe where that system had been successfully adopted.

Mr. Headland. The Author referred to butt-welds between plates of different thi[ck]nesses, and in that connexion it might be of interest to quote results of some tests on joints between two plates, 0·75 inch and 0· inch thick, with the weld made as shown in *Figs. 12.* The tests h been made in connexion with a joint in a 12-foot diameter penst with a working pressure of 75 lbs. per square inch, subject to pressu rises of 30 per cent. As the lower section of the pipe had be inaccessible for welding from the outside, the plates had been as shown, and, basing calculations on the ⅝-inch plate, the maxim stress was 5 tons per square inch, giving an approximate facto safety of 6. Cutting the plates as shown had also enabled dow hand runs to be made in all positions. Two tests had been carr out on 32-tons-per-square-inch material with the welded joints m in the semi-vertical position. In both cases the specimens h

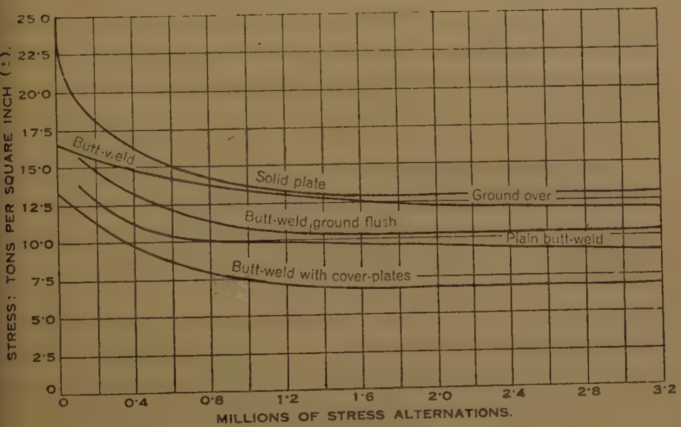
Fig. 13.

fractured at the junction of the weld and the ⅝-inch chamfe plate. *Fig. 13* showed the sequence of welding after tacking 18-inch centres around the circumference, which method had b adopted to prevent undue distortion. In work of that type, it important that a sufficient gap should be left at the root of the to ensure satisfactory initial runs of weld-metal. The relativ large mass of parent-metal compared with that in the weld ha quenching effect which might cause the weld-metal to be somew brittle and liable to cracking, and if that occurred the cracks wo have to be cut out and not welded over. The pipe should supported to avoid excessive distortion due to shrinkage of we metal, but it should not be so rigidly restrained that shrinkage-cra could develop.

Air-vessels for oil-pressure governors were usually of medi size with comparatively thin walls, and, while being air- and tight, were required to withstand working pressures up to 300 per square inch, with a test-pressure, which took the form of a n destructive test, of twice the normal pressure. The tensile stren

as usually sufficient when there was no leakage from the vessel. Mr. Headland.
he cylindrical shell was usually formed slightly elliptical with the ajor axis in line with the longitudinal seam, whilst the joints with ie rounded ends were made with an adequate bevel on the V to ermit proper penetration, and in such a position that excessive resses were avoided. Butt-welds were preferred because, apart om bending stresses and the inferior strength of the lap-joint, ley permitted easier fabrication and proper access for welding. he U-weld, whilst advantageous for X-ray examination, suffered om the defect that the first run of weld-metal had to be it out and the edges of the plates specially machined, as they ould not readily be prepared by oxy-acetylene cutting and

Fig. 14.



rinding. In welded vessels of that type pressure- and temperature-ariations in operation resulted in longitudinal and circumferential reathing stresses, so that inferior welding might fail under such onditions.

Tests up to the value of the test-pressure on vessels of that kind with longitudinal or spiral and circumferential seams showed that he longitudinal and radial deformation followed the calculated alues closely. In cases where the tests had been continued to lestruction, the results had been consistent with those expected from heoretical considerations and tensile tests on the materials up to ind beyond the yield-point. In one case the vessel cracked in the plate at right angles to the direction of the hoop-stress, starting from he centre of the hemispherical end up to half the length

Mr. Headland. of the cylinder, after which the fracture had branched into sections continuing the full length of the cylindrical shell. A from the fact that the branching of the fracture had been du secondary stresses, a satisfactory explanation of the failure woul interesting. The vessel burst, for practical purposes, at the calcul value.

Such tests indicated that, for ordinary static loading, nor methods of design were permissible, and when a good system procedure-control was adopted the welded joint compared more t favourably with riveting in regard to considerations of des construction and cost. When fatigue-stresses were involved, endurance-limit was more uncertain, and he noted that the rang ±11 tons per square inch had already been questioned. endurance-limit of a riveted joint, and even of the plate itself, almost equally uncertain, and it should be pointed out that value of the fatigue-range of a welded joint depended to a l extent on the condition of the plate and the welded joint, as wel on the preparation to which it was subjected. The curves give *Fig. 14* showed some typical test results.

Mr. Jasper. Mr. T. M. JASPER, of Milwaukee, Wisconsin, observed that it mi be said that the employment of ductile autogenous welding pressure-vessel service was about 12 years old in the United Sta It was introduced into the most hazardous service, namely, the cracking industry, the pressure-drums of which operated at 900 and 600 lbs. per square inch continuously. It was several years l that the general codes for vessel-construction had been develope include that method of fabrication. The great advantage of weld then introduced had been its non-liability to leakage and the abilit the weld to resist shock, as well as its being of greater strength t that of the normal plate-materials used in the type of ser referred to.

Without the very progressive attitude of the petroleum-engine who were in need of some new method of construction, the introd tion of welding would have been much slower. Because of t appreciation, their codes for the use of welded-construction w much more complete and easier to apply than any of the ot which had been developed, on account of the great hazard of ser and the longer period over which they had used welded drum service. The joint code of the American Petroleum Institute and American Society of Mechanical Engineers could give the grea help to those desirous of adopting something which would repre safe welded-vessel construction and operation.

The Author had given a very complete resumé of the gen position in the United States, and Mr. Jasper would conclude

ientioning that welding had had the effect of improving the quality Mr. Jasper. nd increasing the number of steels available for pressure-vessel onstruction.

Mr. JAMES MITCHELL observed, with reference to the Author's Mr. Mitchell. emark on p. 623 § that "it is not a difficult matter to assign a efinite figure for the efficiency of a riveted joint," that that result ad only been achieved by the gradually-acquired skill of the workıan, by careful inspection, and by rigid testing. The process desribed in the Paper as fusion-welding was really a variety of soldering, nd considering the enormous number of uses to which soft or tinoldering and hard or brass-soldering, were put, and the very great niformity and reliability of joints so made, there seemed to be good eason for thinking that iron-soldering would, as the skill and xperience of the operators increased and methods of inspection and esting were improved, become equally reliable and equally widepread. As pointed out in the Paper, such joints—as in soldering rocesses generally—presented drawbacks, owing to temperaturetresses, and to the differences which were apt to arise between the naterial of the main body of the plates joined, the solder, and he portions of the plates in its immediate vicinity. With regard o the reference on p. 659 § to the possibility of relieving the internal tresses in such joints by externally-applied static loading, it was robable that every riveted joint had, on completion, a complicated ystem of internal stresses, which were relieved in that way when he joint was stressed. In cases where there was no hardening or lteration of the constitution of the metal in the soldered joint nd its neighbourhood, there would appear to be a good deal to be aid for the relief of stress by means of hydraulic pressure, the ressure being increased as slowly as was practicable, so as to llow a sufficiency of time for the flow of the over-stressed netal. In all other cases, normalizing by heat would be the roper treatment.

Mr. V. E. PULLIN was particularly interested in the Author's Mr. Pullin. emarks on the radiographic examination of welds, and to learn that e—very properly, in Mr. Pullin's view—attached such importance to hat crucial method of test. It might be imagined from the Paper hat the Author attached some importance to the A.S.M.E. assertion hat a suitable penetrameter would ensure the detection of any flaw aving a thickness of 2 per cent. of the plate. Mr. Pullin very much oubted whether the Author accepted that statement of the A.S.M.E. ode, and he rather thought that he would hesitate to confirm it rom his own experience. Mr. Pullin's own researches indicated that

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§ *Ibid.*

Mr. Pullin. the implied assurance that a 2-per-cent.-thick flaw would certainl[y be] shown under the conditions laid down by the American Code [was] fallacious.

Mr. Weinrich. Mr. A. O. WEINRICH, of Mannesmannröhren-Werke, Huckin[gen,] agreed in general with the Author's comments so far as the questio[n of] plate-material was concerned; his firm were of the opinion, howe[ver,] that the manganese- and silicon-contents could be higher than (and 0·20 per cent. respectively. For instance, they had ascertai[ned] that a steel containing from 0·8 to 1·0 per cent. of manganese [and] from 0·25 to 0·30 per cent. of silicon was perfectly weldable by b[oth] the water-gas and the electric fusion-welding processes. Furth[er-]more, it was thought that the question of weldability of material [for] pressure-vessels had not yet been fully investigated, and it was [to] be hoped that there would be further development in that aspec[t of] welding technique. It was of interest to note that in Germany th[ere] was a tendency towards a diminution of the carbon-content [of] weldable material, together with a simultaneous increase of alloy[ing] elements which had the effect of raising the tensile properties.

It might be stated that the German rules for welded pressu[re-]vessels imposed on the German manufacturer certain requireme[nts] regarding heat-treatment, and in consequence metallurgical [and] welding development had been influenced in the direction of obtain[ing] welds of special character in which heat-treatment might reasona[bly] be omitted. The new German rules made allowance for the prog[ress] made in welding technique, in so far as heat-treatment was [not] required for pressures up to 8 atmospheres, with a tensile strength [up] to 42 kilograms per square millimetre and a stress up to 4·25 k[ilo-]grams per square millimetre. When annealing treatment at 650° was carried out the welded seam was allowed a joint-efficiency of (with the proviso that the welded seam had to be subjected to a n[on-]destructive test. In that way no doubt experience would be obtain[ed] and eventually a relaxation of the existing rules would be possi[ble.] That precaution appeared to be necessary, in order to prot[ect] welding from reverses, the consequences of which could be forese[en.]

A series of tests had been carried out in Germany with respect [to] stress-relief by static loading, which, in conformity with the st[ate-]ments of the Author, had the effect of modifying residual stre[sses] and of removing the stress "peaks." However, there were cert[ain] difficulties encountered in that method of relieving pressure-ves[sels] from residual stresses, which mainly occurred due to uneven thickn[ess] of the plates which could not be avoided in the plate-manufact[ure.] As a consequence of that, neither the location of the expansion [nor] the extent of it at a determined static load was a fixed quant[ity.] In that way there could arise critical plastic flow in or beside

ld-seam which, when using steel not resistant to ageing, might lead accelerated ageing of the material under the high temperatures evailing in steam boilers. Mr. Weinrich.

The AUTHOR, in reply, pointed out that the use of mild-steel plates 22 and 24 tons per square inch tensile strength, referred to by r. Dipper, had long been common practice on the Continent in the anufacture of both riveted and welded pressure-vessels, and it had t been the Author's experience that the highest class of welding ork in Germany was any worse or better than that to be found in reat Britain. The Author.

In regard to austenitic welds, the martensitic zone had a lower rbon-content than that found in "cutting hard" martensite. hat zone was certainly composed of material which might be said be martensitic, but which might in fact be structurally composed material in some intermediate phase-condition. In any case, the pe of martensite produced as a result of the rapid cooling of low-rbon steels should obviously be placed in a different category from at associated with steels having a carbon content of from 0·8 to 1 per cent.

The method adopted by the investigators in testing the hardness the martensitic zone in austenitic welds was the Martens scratch ethod, which entailed the scratching of the specimens with a loaded amond attached to a moveable lever. The ratio between the load the diamond point and the width of the scratch gave a hardness gure, which, by means of a factor sufficiently correct for purposes comparison, could be converted to a Brinell number. The width the scratch in the martensitic zone under notice varied between ·01 and 0·012 millimetre, giving calculated Brinell numbers between 40 and 285. The width of a corresponding scratch in tool-steel artensite was 0·006 millimetre, giving a calculated Brinell number 475.

He appreciated the remarks of Dr. Faltus, and could confirm the nportance of ensuring, by regular testing, a supply of electrodes aving a consistently high quality. In criticizing bend-tests, the recise reason for specifying a particular type of test had to be orne in mind, and the bend-test referred to by Dr. Faltus was not tended to demonstrate the quality of fusion between the weld-etal and the parent-metal, but rather to demonstrate the ability f deposited weld-metal, contained in that portion of the joint which as least affected by superimposed runs of weld-metal, to withstand eformation.

He was interested to note that the firm with which Dr. Faltus was ssociated had found X-ray examination indispensable in the manu-acture of high-class welded pressure-vessels. X-ray examination

The Author. had also been discussed by Messrs. Fergusson and Pullin, and it interesting to note the figures given by Mr. Fergusson for the ex cost involved by the manufacturer in complying with the requi ments of regulations such as those issued by Lloyd's Register r Class 1 Pressure Vessels; it was satisfactory to observe that extra cost could be offset against the higher joint-efficiency allow Whilst the Author agreed with the adoption of stereoscopic techni in the X-ray examination of welded joints, especially in the case thick vessels, it was a question of economics rather than of necess and had to be decided by the manufacturers themselves. method of X-ray examination adopted and advocated by Fergusson was excellent, but it would be appreciated that it was the function of inspecting authorities to specify in detail the adopt of any particular technique having special economic advanta The Author agreed with Mr. Pullin that no special significance co be attached to the value implied by the A.S.M.E. Code in respec penetrameters, but at the same time there could be no question t penetrameters should be used in order to provide some indicat on the film that full penetration of the X-rays was being obtain That was especially important in the case of welds which w dressed flush with the parent plate.

Dr. Harris had referred to the practice of allowing a shrinkage- at the root of heavy welds, and he agreed that the weld-metal c tained in the first run was severely quenched and was not in best condition to adjust itself to severe stresses. That was p cisely the reason why the first run of weld-metal should be enti removed from the under-side of the weld, irrespective of whet or not any shrinkage-gap had been provided.

The question of weldability of steel plates, referred to by Harris and Mr. Weinrich, had become a subject for research un the guidance of a special committee set up by the Institute Welding, and there was no suggestion that the mechanical physical analyses of plate-material, detailed on p. 637 §, was inten to serve as a basis for steel-plate specifications. The figures refe to were not uncommon in practice, and material giving such t results had certainly been found entirely suitable for weld Impact-tests had been discussed in some detail in the Paper, and did not intend to imply that there was a definite relationship betw impact- and fatigue-test values. What was suggested, howe was that a material giving satisfactory impact-values might be found satisfactory under fatigue-conditions, but it was neces in the first place to decide what was a satisfactory impact-va

§ *Ibid.*

n that connexion it would be noted that Dr. Harris had brought ut the point that a high impact-value alone was not necessarily a uarantee of a high quality of material. The Author was indebted o Mr. Jasper for his observations, and could confirm that the development of welding in the pressure-vessel industry owed much to the il companies. The Author.

In reply to Mr. Mitchell, he pointed out that few would be preared to agree that fusion-welding bore any resemblance to soldering, nd to suggest that two such entirely different processes of making oints were analogous was undesirable. He could assure Mr. Mitchell hat the skill required for riveting, and the inspection and testing rocedure normally applied to riveted joints, did not approach the tandard laid down for welding.

In regard to the relief of stress in welded joints by means of static oading, it was considered that there was insufficient evidence of the eliability of such a method, and in all cases of important welded ressure-vessels reliance should be placed on heat-treatment carried ut in a properly-constructed furnace. The question of relieving esidual stresses by means of static loading had also been mentioned y Mr. Weinrich, and the Author was interested to learn of the tests arried out in Germany, and of the difficulties encountered due to he uneven thickness of plates.

Mr. Headland put forward some interesting information regarding he welding of various structures such as spiral casings and largeliameter pipe-lines, and referred to a number of points which were lealt with in the Paper. He drew attention to the difficulty of btaining a proper distribution of the load in the case of composite oints involving welding and riveting, and the Author was of the pinion that such joints should be avoided wherever possible. Spiral eams had found some favour on the Continent, but in general it ould be said that the simplicity and reliability of the straight ongitudinal seam, welded under careful procedure control, had led ractically to its universal adoption in the manufacture of welded ressure-vessels. In the case of pipe-lines and long cylinders such s those referred to by Mr. Headland, there might well be special dvantages to be derived from the adoption of spiral seams, and it was of interest to note that the system had been used in the construction of several pipe-lines in Europe.

The form of joint indicated in *Figs. 12* (p. 509) was not considered satisfactory in that no attempt had been made to reduce the thickness of the thick plate to that of the thin in way of the weld, and, whilst the results of the static tensile tests given by Mr. Headland appeared to be good, it should be noted that the ultimate fracture occurred at the junction of the weld at its weakest section, and it

The Author. was considered that fatigue tests on samples of that type wou leave little doubt in regard to its deficiency.

In regard to butt-welds, the choice between the V and the welds was a matter for the manufacturer. The Author had point out the advantages of the latter type, and in regard to Mr. Hea land's remarks, it might be said that, for high-class welding-wo plate-edges should always be machined prior to welding, and that both types of joint the first run of weld-metal at the root of the we was generally contaminated and should be removed by cutting.

Paper No. 5109.*

"Kincardine-on-Forth Bridge."

By John Guthrie Brown, M. Inst. C.E.

Correspondence.

Mr. R. D. Brown wished to call attention to a very important question that was often overlooked in the design of swing-bridges: had any provision been made for the repair and renewal of the rollers, and of the upper and lower roller-paths? Mr. Brown.

That question had arisen acutely on the Manchester Ship Canal, when it had been found necessary to repair or renew the upper and lower paths and the rollers of several swing-bridges. The swinging weight of those bridges had ranged from 640 to 1,800 tons, and the cost per bridge of the repairs and renewals of paths and rollers had varied from £11,712 to £30,589.†

In every case it had been found necessary to raise the bridge temporarily, and good foundations for the support of the lifting jacks had been found at no great depth. In the case of the Kincardine bridge, however, the foundations would be deep and costly. Some of the problems which would have to be solved in carrying out those repairs were:—

(1) At which panel-points was it best to raise the bridge, (*a*) so that no local damage was caused to the structure, (*b*) so that the minimum reversal of stress in the members was caused, and (*c*) so that a minimum of additional structural steelwork need be provided?

(2) How could foundations be provided capable of sustaining the weight of the bridge-superstructure when it rested upon the lifting jacks?

(3) What provision could be made for guarding the bridge from accidental damage by shipping when poised upon the jacks?

(4) What provision could be made for traffic while the repairs were being carried out? Which traffic would take precedence during the repairs—the traffic using the bridge or the traffic using the waterway?

(5) What method of repair could be adopted so that the time required might be reduced to a minimum?

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* Journal Inst. C.E., vol. 5 (1936–37), p. 687 (April, 1937).

† R. D. Brown, "The Raising of Barton Swing-Aqueduct and the Renewal of Paths and Rollers." Selected Engineering Paper No. 67, Inst. C.E., 1929.

Mr. Brown.

It was easy to dismiss those problems and to leave their solutio to some later stage, but it was obvious that the expenditure of small sum at the time of building would make provision for thos requirements and would avoid the spending of a large sum in th future. In the case of accidental damage, or of damage due to enem action, it would be possible to repair the bridge in a fraction of t time that would otherwise be required.

The Author.

The AUTHOR, in reply, stated that Mr. Brown had very proper drawn attention to the importance in designing swing-bridges keeping in view the repairs and renewals that might arise in futur

Particular attention had been given in the design of the swin span of the Kincardine bridge to provide duplicate means of operatio wherever possible, so as to prevent a complete breakdown occurrin For the same reason the design was arranged so as to permit repai and renewals to the rollers and tracks to be carried out with si plicity and expedition.

Should an extensive overhaul of the swing span be necessary the future, then it would be essential to carry it out with the sp in the position open to river-traffic. In that position the structu would be adequately protected from shipping by the timber fenderin which would also be available to assist in supporting the bridge required.

With the bridge closed to road-traffic, vehicles would requi temporarily to use the Stirling bridge, as had been the case befo the construction of the bridge at Kincardine. No such alternati was available for shipping, however, which had therefore to recei uninterrupted use of the river during repairs to the structure.

CORRESPONDENCE
ON PAPERS PUBLISHED IN
JUNE 1937 JOURNAL.

Paper No. 5085.[1]

"The Reconstruction of the Chester—Holyhead Road near Penmaenmawr, North Wales."

By Cecil Lee Howard Humphreys, T.D., M. Inst. C.E.

Correspondence.

Mr. T. W. Moran observed that, apart from the major engineering difficulties, there had been the ever-present element of danger, which had been increased by the proximity of the existing road above and the railway-line below. The two diversions had improved the traffic-conditions around the headlands out of all recognition, and it was to be hoped that at some future date further widening might be done east of Penyclip and east of Penmaenbach, as certain parts of the road were much too narrow for the volume of traffic that passed during the summer. **Mr. Moran.**

The question of leakage through the tunnel-lining, referred to on pp. 70 and 80 §, was a recurring difficulty in vehicular tunnels in water-bearing strata, a very minute quantity of percolation being sufficient to disfigure the interior finish. He rather doubted if continuous concreting would provide a completely watertight lining, as construction-joints were not the only sources of infiltration. If water were present in the rock whilst concrete was being deposited, some porous patches would be almost inevitable, and subsequent pressure-grouting was not always effective in sealing them. There was also the question of the initial shrinkage of the concrete on drying out, which would amount to about ½ inch in 100 feet. The extrados of the lining would be completely restrained by the rough rock-face, whilst the intrados would be relatively unrestrained. The concrete at the intrados would cool off and dry out fairly rapidly, and contraction-cracks would probably occur unless joints of some sort were provided.

Temperature-cracks (pp. 77 and 80 §) were another possibility,

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[1] p. 28, *ante*. (June, 1937.)

§ Page numbers so marked refer to the Paper.—Acting Sec. Inst. C.E.

Mr. Moran. and from observations made elsewhere he would anticipate that t seasonal range of temperature on the North Wales coastal road w quite sufficient to cause hair-cracks. The temperature-conditions the Mersey tunnel (to give only one example) were much more stab but surface crazing was visible in the interior finish, particularly ne the entrances; that might be due to the interior surface being subje to greater fluctuations of temperature than the extrados of t lining. The probability was that the hair-cracks which admitt percolation through a concrete lining were due to the combin effect of initial shrinkage and subsequent cold-weather contracti rather than to the effect of either taken separately. The temperatu of the mix at the time of placing the concrete was an important fact in both cases. A familiar feature of concrete-construction was th concrete placed in warm weather was much more liable to devel contraction-cracks than concrete placed in cold weather.

He had had some experience with various forms of cement rende ing for the purpose of providing a watertight finish in tunnels, b he had come to the conclusion that any rendering would fail in cour of time, either from cracking under secondary stresses or fro becoming subjected locally to the full hydrostatic pressure. C p. 70 § the Author indicated that the best solution lay in the use of a interlining, and with that Mr. Moran fully agreed. During the pa 2 years he had examined that aspect of tunnel-construction at som length in collaboration with Mr. C. B. H. Colquhoun, M. Inst. C.E and as a result a special type of cavity lining had been evolve That consisted of an interior secondary lining spaced about 1 inc from the primary lining, so as to leave a cavity all around for natur drainage. The secondary lining was formed of reinforced " Gunite and was designed as a self-supporting arch, carrying no load but i own weight. It rested upon longitudinal skewbacks projecting fro the primary lining, and was not dependent upon hangers which mig corrode away. Frequent openings were provided in the skewbacks t lead away any moisture which might seep through the primary linin and adequate provision was made for relative movements, and for t relief of all secondary stresses. When further experience had bee gained in the use of secondary linings, it was quite probable that a overall reduction in the duration and cost of tunnel-constructio might be obtained, as under existing circumstances the measur required to secure complete watertightness in the primary lining we often both tedious and costly.

A feature of the diversion works as a whole was the way th blended naturally into their setting, and that applied not only to t

§ *Ibid.*

viaduct and tunnels, but also to the subsidiary work, such as the facing of the rock cuttings. Mr. Moran.

With regard to the Author's remarks on the lighting of the Penmaenbach tunnel (p. 71 §), he had noticed a somewhat similar effect elsewhere. In the daytime, the motorist might experience an almost instantaneous change from brilliant sunshine to diffused artificial light, which was rather disconcerting. During the past 2 or 3 years several new forms of lighting had been introduced (for example, sodium- and mercury-vapour discharge-lamps), which gave excellent results for street illumination. It would be of interest to have the Author's views on their suitability for the illumination of vehicular tunnels.

Mr. H. F. Wilmot, commenting on the Author's remarks concerning the concrete lining to the tunnel, observed that construction-joints, but not necessarily expansion- or contraction-joints, were generally considered from various viewpoints. In concrete-work, the description "contraction-joint" was much more accurate than "expansion-joint," since the stresses in the concrete on setting were far more severe than those normally encountered due to temperature-variation. Such joints were essential only where the work was "free"; stresses then developed without restraint, and they naturally tended to relieve themselves at a weak spot, with the result that unsightly cracks occurred. Mr. Wilmot.

Where work was constrained, however, as when forming a lining over such an ideal surface as a rough rock-faced tunnel-excavation (and of plutonic rock at that), it was clear that no cumulative movement could take place, and therefore the stress in the concrete remained evenly distributed over the whole of the tunnel-lining, relieving itself where it could by developing hair-cracks, which were harmless and were not unsightly.

That should be the underlying principle also for all road-surfaces. Provided that the foundation was thick and well enough anchored to ensure that the surface-stresses were distributed uniformly, no contraction-joints should be necessary. That clearly presupposed that the wearing surface was adequately attached to the foundation, and applied especially to concrete surfaces. An acknowledgement of the principle should have evolved cross-trenches at intervals to form "cross-beams" with haunches in the foundations of roads, and by so doing would have reduced the overall thickness of ordinary mass-foundations, and incidentally would have tended to cheapen the construction of new roads. Its success would be certain.

Types of surface-construction had been evolved to distribute

§ *Ibid*

Mr. Wilmot. the temperature-stresses uniformly over the foundation, but he c not know how far they had achieved recognition; their success w bound ultimately to depend on the degree to which the foundati achieved rigidity, in order to give them the necessary constraint.

The Author. The AUTHOR, in reply, observed, with regard to the water prese in the rock, that while concrete was being deposited there was n enough water, in his opinion, to cause porous patches.

With regard to shrinkage of the concrete, it was quite possible th some hair-cracks were inevitable, but observation actually show that the real trouble lay in the construction-joints. Had those joi not existed he was of opinion that the amount of water which wou have shown on the surface would have been trifling.

Regarding the illumination of vehicular tunnels, he was of opini that the most satisfactory form which had yet been devised was th which was used in the Mersey tunnel, where the lights were set deep recesses covered with ground glass, so that glare was eliminat while a high general intensity of lighting was obtained. He wou have liked to adopt a somewhat similar system for the Penmaenba tunnel, but it was not considered that the length of the tun warranted the expense, and it was also felt that the form of co struction of the tunnel did not lend itself so readily to the use of de fittings set into the concrete.

Paper No. 5113[1].

"The Flow of the River Severn, 1921–36."

By Professor STEPHEN MITCHEL DIXON, O.B.E., M.A., B.A.I., M. Inst. C.E., GERALD FITZGIBBON, B.A., B.A.I., and MICHAEL ANTHONY HOGAN, D.Sc., Ph.D., M. Inst. C.E.

Correspondence.

Mr. A. A. BARNES, from his knowledge of the gauging-site, regretted that the Authors had not considered it worth while to correlate surface water-slopes at the site with the discharge-measurements. They stated on p. 94 § that "as the channel is neither uniform nor straight, and the bed-slope is irregular, no attempt can be made to compare the observed relations between surface-slope and discharge with any of the formulas for calculating the discharge of open channels." That statement was true in regard to the total length of 2½ miles from the Severn bridge gauge to the Bewdley bridge gauge, but at the site itself there existed a straight and uniform reach about 300 yards long with a relatively smooth rock bottom, the gauging-section being in the centre of that portion (p. 85 §). He had visited the site on five occasions during July and August, 1937, and he had been able to obtain the exact water-slopes for the low summer stages. Depth had been taken on each occasion down to the water-surface with a steel tape at ten points in a length of 1,000 feet, the datum points being very accurately-levelled flat-topped nails driven horizontally into overhanging tree-trunks; it had been found possible on calm evenings to register the water-level to $\frac{1}{16}$ inch. The results were given in Table A, and when plotted produced the water-surface slopes i given in the last line of Table XI. Unfortunately, no floods had occurred during July and August, 1937, and the range covered was slightly over 1 foot (from 5·40 to 6·42 feet on the standard gauge at the site); sufficient information had, however, been obtained to prove that the site was ideal for a study of water-slopes, and that fact made the absence of a complete series of readings all the more regrettable. Mr. Barnes.

A formula developed by Mr. Barnes which had not yet been published made it possible to calculate the discharge-curve from the above water-slopes so far as those had been taken. It would be

[1] p. 81, *ante*. (June, 1937.)

§ Page numbers so marked refer to the Paper.—ACTING SEC. INST. C.E.

Mr. Barnes. seen that the temperature of the water had been observed in e case, so that having the values of A, m, and i (denoting the cr sectional area, the wetted perimeter and the surface-sl respectively), the velocity v, and thence the discharge, could calculated from the formula :—

$$\log \log \frac{vmp}{\mu} = \log N + \beta \left[\log \log \frac{g}{2} \cdot m^3 \left(\frac{\rho}{\mu}\right)^2 . i \right]$$

or $$\log \log R_m = \log N + \beta(\log \log B_m)$$

or $$\log \log (\text{Reynolds No.}) = \log N + \beta \; \{\log \log (\text{Barnes No.})\}$$

in which N was the coefficient of roughness, and the accompany

TABLE XI.—READINGS IN FEET OF WATER-LEVELS AT THE GAUGING-ST

Temperature of water: °F.		65	68	63	65	66
Date		21 Aug., 1937.	15 Aug., 1937.	11 July, 1937.	16 July, 1937.	17 Jul 1937.
Point.	Distance from gauge: feet.					
A	+531	5·414	5·653	5·713	6·338	6·464
B	+259	5·405	5·647	5·697	6·313	6·436
C	+183	5·402	5·634	5·696	6·307	6·428
D	+139	5·408	5·636	5·699	6·313	6·431
E	+106	5·399	5·631	5·690	6·300	6·425
F	+ 31	5·401	5·632	5·693	6·300	6·427
Gauge	0	**5·40**	**5·63**	**5·69**	**6·30**	**6·42**
H	−102	5·393	5·623	5·687	6·289	6·409
J	−260	5·393	5·613	5·681	6·279	6·395
K	−354	5·387	5·614	5·670	6·277	6·394
L	−494	5·388	5·614	5·671	6·265	6·386
From plottings	+500	5·416	5·651	5·713	6·337	6·459
	−500	5·384	5·609	5·667	6·263	6·382
Fall in 1,000 feet		0·032	0·042	0·046	0·074	0·077
Surface-slope i		0·000032	0·000042	0·000046	0·000074	0·00007

index $\beta = 1{\cdot}2580 - 0{\cdot}4343N$. Due chiefly to some very accur work with surface-slopes by Mr. W. N. McClean on the Mucomir at the outlet to loch Lochy, it was possible to state that the value N for that type of river was 0·71, and therefore that $\beta = 0{\cdot}95$. should be observed that the above basic equation could be rep sented by a straight line $y = \log N + \beta x$, and it was found, in sp of the very different values of m and i in the two sets of experimen that all of Mr. McClean's fifteen observations and the five on the ri

ern lay exactly on one common straight line when plotted with Mr. Barnes. above co-ordinates. The value of log v could therefore be culated by taking the correct values of log ρ/μ for the observed nperatures of the water, as given in Table XII, and by denoting $g/2$ by 1·2067. The results so obtained were given in Table XII, ilst in order to allow the Authors to compare those calculated charges with those found by them with current-meters, the ures had been reduced to a common average temperature of ° F., for which log ρ/μ was 4·8837. For higher gauge-readings e values of A and m were already known from the section, so that

BLE XII.—DISCHARGES CALCULATED FROM OBSERVED READINGS OF m, i, AND TEMPERATURE.

Date: 1937.	Gauge: feet.	Area: square feet.	m: feet.	i	Water temperature: °F.	Log ρ/μ: seconds/square feet.
1 Aug.	5·40	578	4·05	0·000032	65	4·9454
5 Aug.	5·63	613	4·27	0·000042	68	4·9629
1 July	5·69	622	4·32	0·000046	63	4·9334
6 July	6·30	710	4·86	0·000074	65	4·9454
7 July	6·42	728	4·97	0·000077	66	4·9513

uge: eet.	At observed temperatures.				At 55° F.			
	log log Bm	log log Rm	v: calculated feet per second.	Q: calculated cusecs.	log log Bm	log log Rm	v: calculated feet per second.	Q: calculated cusecs.
5·40	0·92558	0·73055	0·667	386	0·91917	0·72447	0·647	**374**
5·63	0·93686	0·74126	0·828	508	0·92883	0·73365	0·797	**489**
5·69	0·93665	0·74108	0·871	542	0·93163	0·73629	0·850	**529**
6·30	0·95552	0·75900	1·286	913	0·94954	0·75331	1·248	**886**
6·42	0·95832	0·76166	1·345	979	0·95181	0·75547	1·302	**948**

hen further values of i could be obtained the complete discharge-irve could be calculated by means of the above formula. It would e of interest to compare the curve with that given by the current-eter observations, seeing that the set of ten water-levels for each alue of i could be taken by one man in only 35 minutes with no quipment beyond a steel tape, as compared with probably $2\frac{1}{2}$ hours t least with several observers for one complete set of current-meter bservations.

His levellings had disclosed the fact that a serious error existed in ie standard gauge-post at the site. That post was in four lengths of ast iron set in concrete and graduated in tenths of a foot, but due

Mr. Barnes.

to some unknown cause the tops of the posts did not form a continu scale. They should read 8, 10, 13, and 16 feet, but the actual hei were as follows :—

Gauge post : feet.	Datum level : feet.	Error.
8·00	8·000	Taken as standard.
10·00	9·972	$\frac{3}{8}$ inch low.
13·00	12·773	$2\frac{3}{4}$ inch low.
16·00	15·852	$1\frac{7}{8}$ inch low.

The posts did not seem to have sunk or to have been disturbe any way, and hence it had to be supposed that a permanent erro $2\frac{3}{4}$ inches had always existed for levels between 10 feet and 13 and that the highest post had also always been low. He wond how that would affect both the upper portion of the stage–disch curve and the relation between water-levels at the gauging-site at Bewdley as shown in *Fig. 6* (p. 95 §).

Mr. Griffith.

Mr. W. M. GRIFFITH was surprised that it should have been c sidered necessary to select a river site for the purpose of study the best method of measuring and recording discharges. experience indicated that the best method would depend on the riv the site, the local conditions, the purpose for which the discha were required, and the staff available to obtain the necessary d To select a site where the conditions were ideal and presented fe difficulties than was normally met with did not appear to prov the most valuable data on the point under investigation.

An executive engineer holding charge of an Irrigation Divisio the United Provinces of India would normally, as a matter of rout be responsible for computing the daily flow of from twenty to thi different channels carrying discharges ranging from perhaps sev thousand cusecs to, say, 10 cusecs. To obtain the necessary d daily readings of gauges were taken, and at most one discharg each channel per month was observed by a subordinate. To comp the discharges from those data the engineer had a draughtsman usually had but a slight knowledge of hydraulics and a head clerk had none. The channels themselves, unlike the site selected on river Severn, had erodible beds, which were constantly changing the discharge was not a fixed function of the water-level and mi vary for the same water-level by anything from 5 to 30 per cent no great length of time, due to silting or clearance. The prob presented some difficulty, but a method had been adopted which g good results provided that the data were correctly reported.

§ *Ibid.*

Mr. Griffith.

Mr. Griffith had at one time been responsible for gauging and omputing the daily discharges of the Sarda river at Tunakpur, a te some 3 days' march from headquarters, in the Terai jungle. he river-velocities were too high to allow of the use of a boat, and ie only method of crossing the river was on an elephant, which at mes had to swim. That river-bed changed its section annually 1 the flood-season, but it was possible to obtain a reasonably correct age–discharge curve for the winter flow from the few discharges it 'as possible to observe in each season. One year in which only one ischarge had been observed presented some difficulty, but it had een found possible by careful reasoning to prepare a reasonably orrect stage–discharge curve for computing the daily records of flow or that year from the daily gauge-readings which had been recorded.

The problem of the best method to be adopted often depended ery largely on the purpose for which the gauging was required. 'or example, if the information were required for computing the otal annual run-off for the purpose of comparing it with the rainfall, record of the daily discharges was necessary, but for that purpose t was not necessary to know the discharge on any one day with reat accuracy, provided that the sum of the daily totals was reason-bly accurate. If, then, the method adopted were an approximate nethod giving uniformly a plus or minus error on the three hundred nd sixty-five observations of the year, a great deal of the error would alance out, and the annual total would be more accurate than the laily discharges.

On p. 90 § the Authors noted, with reference to *Fig. 4*, that the curve f cross-sectional areas was convex to the depth-axis. The area-urve would theoretically only be a straight line if the sides of the iver were vertical or parallel, and were not inclined. Actually as lrawn that area-curve was shown as a straight line from about auge-level 15·0 to gauge-level 19·0. It was not clear how that curve ould be obtained, as the sides in that range could hardly be vertical n the light of the Authors' reference to the river overflowing its anks into the shallow areas on either side, and also in view of the hange in slope of the hydraulic-mean-depth curve, which indicated rapidly widening cross-section within that range. Further elucida-ion of that point would be welcomed.

The Authors noted that the velocity-curve was concave to the depth-xis. It was possible to define the characteristics of that curve ather more completely, as the velocity-curve would approximate o the equation $V = C\sqrt{D - d}$, where V denoted the mean velocity, D denoted the gauge-reading, d was approximately the gauge-

§ *Ibid.*

Mr. Griffith. reading at which the velocity became zero, and C was the coeffic for the gauging-site. Judging from *Figs. 4* and *5* (p. 91 §) the va of d was about 5·25, so that the equation might be written

$$V = C\sqrt{D - 5{\cdot}25}, \text{ or } C = \frac{V}{\sqrt{D - 5{\cdot}25}}.$$

To find the value of C one value of D and one of V were requi Taking a value of $D = 15{\cdot}0$, the velocity for that value of D scaling on *Fig. 4*) was found to be approximately 4·4 feet per seco The equation might then be written

$$C = \frac{4{\cdot}4}{\sqrt{15{\cdot}0 - 5{\cdot}25}} = 1{\cdot}41,$$

so that the equation to the velocity-curve would become appro mately

$$V = 1{\cdot}41\sqrt{D - 5{\cdot}25}.$$

The equation for the stage–discharge curve followed directly fr that of the velocity-curve, and the stage–discharge curve wo therefore approximate to the equation

$$Q = 1{\cdot}41 \,.\, \Delta\sqrt{D - 5{\cdot}25}$$

where Q denoted the discharge and Δ denoted the sectional area any gauge-reading D. If values of Δ were scaled from *Fig. 4* a substituted in the above equation the resultant curve would be se to approximate fairly closely to the value of the stage–discha

TABLE XIII.

Gauge-reading, D: feet.	$(D - d) = (D - 5{\cdot}25)$: feet.	Cross-sectional area, Δ: square feet.	Calculated discharge, $Q = 1{\cdot}41 . \Delta . \sqrt{D - 5{\cdot}25}$: cusecs.	Value of Q_1 scaled from stage–discharge curve, *Fig. 5*: cusecs.	Difference: cusecs.	Percenta of differen
Col. 1.	Col. 2.	Col. 3.	Col. 4.	Col. 5.	Col. 6.	Col. 7
18·0	12·75	2·600	13,100	13,665	−565	−4·1
16·0	10·75	2,200	10,200	10,600	−400	−3·8
14·0	8·75	1,860	7,760	8,000	−250	−3·1
12·0	6·75	1,520	5,580	5,500	+ 80	+1·6
10·0	4·75	1,215	3,770	3,600	+170	+4·7
8·0	2·75	900	2,105	2,050	+ 55	+2·7

curve in *Fig. 5*. Table XIII was given to illustrate that res Col. 4 gave the discharge Q, calculated by the equation

$$Q = 1{\cdot}41 \,.\, \Delta \,.\, \sqrt{D - 5{\cdot}25},$$

§ *Ibid.*

lues of Δ (col. 3) being scaled from the cross-sectional-area curve, Mr. Griffith.
ig. 4. Col. 5 gave the discharge-values Q_1 as scaled from the stage–scharge curve, *Fig. 5*. The difference and percentage of difference as given in cols. 6 and 7.

That example was given as one method of obtaining an approximate age–discharge curve if only one observed discharge were available.

The work described by the Authors was in effect that of rating a auging-site, at which, owing to fortunate circumstances, the discharge was an unvarying function of the water-level. Under such rcumstances, time and opportunity permitted a very accurate ting. Those were ideal conditions which were unfortunately ldom met with in practice at sites where the discharges were quired to be known.

When the function altered slowly, due to gradual silting or scour f the bed, it was possible to prepare a normal stage–discharge urve based on average conditions. Values from that curve could e corrected by a percentage factor obtained by periodic discharge-bservations. That was the method adopted by the irrigation ngineers referred to above. The method would give reasonably orrect results provided that the discharge was fairly stable, but it as unreliable if the discharge varied greatly in the interim of the eriodic discharge-observations. Where the water-level varied reatly, or where the function changed rapidly and irregularly, the ethod failed. It also failed where tidal flow affected the water-level, r if the water-level were affected by sluice-regulation below the site, s the discharge ceased to be a function of the water-level.

He was responsible for gauging the river Great Ouse and its ributaries, and he found that at none of the sites where a record of he daily discharges was required to be known was the discharge a table, or with one exception even a computable, function of the vater-level, on account of :—

(*a*) tidal action in the lower reaches,
(*b*) the effect of dams and sluices and the rapid growth of weeds in the upper reaches.

That fact had made the problem difficult, but by obtaining daily t the gauge sites both the gauge-reading and the timing of a surface-loat, it had been found possible to compute the daily discharges of he river Ouse and its tributaries at ten such sites for the past 2 years vith sufficient accuracy to supply the necessary information required or designing proposed improvements to the drainage-system.

The method of computing a river-discharge from the timing of urface-floats was old, but was not well understood and was little sed in England, mainly, he believed, on account of lack of accurate

Mr. Griffith. knowledge of the relationship existing between surface- and mo velocity, and of knowledge of how and under what conditions th relationship changed. The Authors, in using floats for comput the discharge, had assumed that the relationship would hold only fo width of 10 feet across the section, and therefore timed from thirty sixty floats across the section, which was laborious. It was, howev not illogical to assume that the relationship would hold at least constantly across the section as vertically, because the velocit varied more consistently and regularly across the section th vertically. Accepting that, it was clear that the timing of one fl should give the required information for computing the me velocity, provided that the relationship be known.

If the float were timed on the line of maximum surface-velocity, t relationship between its timing and the mean velocity then should l consistent for any water-level, and that relationship could be fou by rating observations if time and opportunity permitted. Ba had evaluated that function in his well-known formula

$$\frac{V_m}{V_s} = \frac{1}{1 + \frac{25 \cdot 4}{C}},$$

where V_m denoted the mean velocity and V_s denoted the maximu surface-velocity, but his evaluation was empirical, based sixty-one series of gaugings, and although his value was govern by the hydraulic mean depth, there was evidence that it was n fundamental but was only approximately true within limits, and a normal symmetrical section. That fact had been recognized, a in using that relationship the normal practice was to divide irregular section into two or more regular sections by imagina vertical planes, and to accept Bazin's relationship of $\frac{V_m}{V_s}$ for each those sections separately. That practice necessitated running flo in each of the sections to compute the net resultant flow.

For the gauge-readers of the Great Ouse Catchment Board to ha to run more than one float in the cross-section presented difficulti The necessity of running more than one float only arose in time flood when the river spilled over its banks, because it was alwa possible to find a site where within its banks the river had a regu and fairly symmetrical section. That fact, however, did not he matters, as the correct assessment of the flood-discharges was mo important than that of the normal discharges.

In the course of a Paper [1] published by The Institution in 1927

[1] "A Theory of Silt and Scour." Minutes of Proceedings Inst. C.E., vol. (1926-27, Part 1), p. 246.

had claimed as a hydraulic law that in a flowing section having a stable perimeter the mean velocity in the vertical plane would vary across the section approximately as the square root of the depth. Mr. Griffith.

Accepting that law, it was possible to calculate the values of the maximum surface-velocity in those additional sections in terms of that of the normal section, and so to save the necessity for running more than one float, even in flood-times when the section became irregular. The method was, in effect, an amendment of Bazin's ratio, to suit irregular as well as regular sections.

Discharge-observations taken in those flood-sections had proved that the assumption was justified provided there was reasonably parallel flow, and that by that amendment it was possible to calculate with reasonable accuracy the discharge of an irregular flood-section from the timing of one float run in the line of maximum surface-velocity in the normal main river-section.

Check discharges taken by current-meters at the G.O.C.B. sites showed that the method referred to above of gauging discharges by a single float gave reasonably accurate results, provided that the maximum surface-velocity was accurately assessed by the float, and that the main error was not in computing the value of $\frac{V_m}{V_s}$, but in gauging the surface-velocity by the float. Some results, shown in Table XIV, would illustrate the point.

TABLE XIV.

Serial No.	River and site.	Maximum surface-velocity as observed: feet per second.		Difference.	Percentage of difference.	Remarks.
		By meter.	By float.			
1	River Ouse at Brownshill Staunch.	1·96	2·29	0·33	14·4	—
2	River Ouse at Bedford.	1·66	2·01	0·35	17·3	—
3	River Ouse at St. Neots.	2·30	2·60	0·30	11·50	—
4	River Little Ouse at Thetford.	1·25	1·66	0·41	24·6	Discharge run partly blocked by weeds—results unreliable.
5	River Nar at Wormegay high bridge.	1·65	1·64	0·01	0·6	—

Mr. Griffith. One of the principal errors in float-velocity observations was th due to the effect of wind on the float. Another possible error appear to be the effect of surface-tension; he was at present investigati that matter, and Messrs. George Kent, Ltd., were making a sm portable anemometer to his own design, by means of which it w hoped to eliminate a large part of that error. Having eliminated th variable error, it was hoped to find a means of reducing the error d to other causes.

Some observations made in the tidal river indicated that Bazi relationship would hold for the ebb-tide, but would give too low result for the flood-tide. That matter was also under investigati and he hoped to be able to embody the results in a general Paper measuring discharges by a single surface-float at a later date.

Mr. Lacey. Mr. Gerald Lacey, of Roorkee, India, observed that it was interest to note that, despite the fact that the meters were accurat rated, the discharge by the cup-type meter was, taking an averag 1·5 per cent. greater than that by the screw-type meter. If it we correct to state that the true discharge lay between the two ty of observation, the fact was of considerable value. In India the u of current-meters in measuring canal-discharges was steadily i creasing. It was evident that in future, when discharges we quoted, the type of current-meter should be mentioned, and it al should be stated whether each section was integrated or wheth the single- or two-point method was adopted. It would also be interest to know whether, when velocities were measured at o point, comparatively remote from the bed (say at 0·60 of the dept the two different types of meter still displayed the same discrepanc

In the United Provinces in India, and also in the Bombay Pre dency and Sind, adjustable velocity-rods were still largely employ in canal-discharge observations. He had evolved an improved te scopic type which was employed in the United Provinces, whilst Bombay and in Sind a modified type known as the Lacey-Dap rod was employed. The rods were formerly known as Cunningha rods, but their use could be traced to an earlier date than Colo Cunningham's classic experiments on the Ganges Canal, bundl of rods having been used in gauging the Tiber more than a centu ago, and poles having been used in Holland. The advantage rods was that they gave with considerable accuracy the mea velocity in a vertical, and thus saved computation. Also, since ro swept throughout the selected run, they were not constantly expos to eddies which might characterize one point at a cross-section a river.

The reference to surface-float observations was of interest. It w evident that in sudden floods the only records available might

loat-observations; *Fig. 5* (p. 91§) showed that two out of the three Mr. Laoey. loods had been estimated in this manner. In India it frequently ccurred that the only means of computing a river flood-observation was by means of one or two solitary central surface-observations; n occasion record floods had been reported, and found their way nto hydraulic literature, when as a fact the floods had merely been stimated from cross-sections, an estimate of the flood-plane from loating debris stranded near high water mark, and a guess as to the orrect value of Kutter's rugosity coefficient.

In India for many years to come reliance in river flood-discharges vould of necessity be placed on central surface-velocity observations. n *Fig. 19* (p. 111§) the Authors had plotted values of the float-oefficient for different gauge-readings. A large number of float-bservations had, however, been taken at different sections. With fairly regular section it should be possible to correlate the central r maximum surface-velocity with the mean velocity. He would e grateful, provided that the data were available, if another liagram showing the central surface float-coefficient could be given. n his experience the central surface-velocity on a day without a igh wind gave very consistent results. When the banks were opped to such an extent as to involve a reduction in the hydraulic nean depth, the discharges for the bank portions would require to e separately computed.

Mr. David Lloyd suggested that the Authors should differentiate Mr. Lloyd. etween discharge as a rate and discharge as a volume, which were sed synonymously, whereas in places volume discharged, embracing rue run-off and seepage, was implied. Discussing the annual period, he Authors (following American practice in snow-regions) selected hat commencing on the 1st October, giving as their reason that round-water storage would then be a minimum. It followed that or purposes of annual comparison the Authors assumed that all the early minima were of similar value; in other words, that the yearly ow-level of the water-table had a uniform value. Was, however, hat assumption likely to be true? It would be of interest if the uthors would indicate the basis of that deduction, since data of vater-level tables indicated saturation (a constant zero) at the end f February; also, in general, that was the period of commencement f sub-average flow. Those points had weight with an investigator f problems of discharge from catchments of under 50,000 acres.

The hydrograph of the flood of 6–22 February, 1928 (*Fig. 3*, . 99 §), presented a rapid decline. It might follow that it would be lifficult to support a view that appreciable run-off from the preceding

§ *Ibid.*

Mr. Lloyd. period had been carried over into the period commencing on t 21st February, and it was bound to be concluded that that hydr graph was not typical of winter months. Perhaps the Autho would amplify their statement (p. 105 §) that negative loss "was d to the discharge of rain that had fallen in the previous month Some hydrologists would regard that negative loss as condensatio and would find reason to believe it due to occult rainfall, at least part. Undoubtedly there was on the average a time-lag betwe rainfall and discharge. The special feature of the flood of May–1 June, 1924, was its gradual decline, and it would appe that about 1½ inch of run-off had been carried into June. Such carry-over would not be consistent with a definite (see p. 105 relation between run-off and rainfall for that month. It was ther fore suggested, as the included hydrographs were of floods of dime sions likely to affect a monthly relation, yet not typical, that it wou be of value to ascertain an average hydrograph in order to adduce t average lag between rainfall and run-off. Alternatively, it seem possible to calculate the average lag by mathematical treatme proceeding from the histogram of the drainage-area (derived by equ water-travel lines) similar to discharge-expressions recent elaborated for stream-flow by Mr. R. T. Zoch.[1] It might then possible to correlate monthly rainfall and discharge in the period lag.

There was no doubt that the causes which combined to produ the variations in loss were vitiated on a large river drainage-ar by the time-lag, but the data obtained at a smaller catchment we not so affected, and appeared capable of analysis. Such a corr lation-analysis supported the hypothesis that loss was a function several meteorological elements. Professor R. A. Fisher's method of inductive inference showed that there might be several simp relations involved. Would the Authors not consider that the pro pect of ascertaining the true relations was in danger of being befogg by the plethora of speculation or prejudice regarding the water-ye and augmented monthly flows? All such speculation was direct towards establishing spuriously the fact that loss was a function rainfall alone, when plainly it was a joint function of several element as was demonstrated in the case of the Vyrnwy catchment-area.[3]

Mr. Williams. Mr. G. BRANSBY WILLIAMS observed that the description of t

§ *Ibid.*

[1] "On the relation between Rainfall and Streamflow." *Monthly Weath Review*, vol. 64 (1936), p. 105.

[2] "Statistical Methods." London, 1930.

[3] D. Lloyd, "Rainfall and Loss over the Vyrnwy catchment-area." Quarter Journal Roy. Meteor. Soc., vol. 62 (1936), p. 219.

methods of measuring the flow adopted would be of value to those who had to make similar investigations elsewhere. Since it was often impracticable, however, to make use of instruments such as those described, and the rates of discharge of rivers had then to be ascertained by simpler means, it would have been useful for such cases if the gauging-station could have been placed where there was a sufficient reach of river of uniform section to enable a comparison to be made of results obtained from formulas for velocities based on surface-gradients and cross-sections of channels. Mr. Williams.

The Parliamentary Joint Committee on Water Resources and Supplies, in their recent Report, had recommended that the Catchment Boards set up under the Land Drainage Act of 1930 should be given the duty of gauging rivers on their tributaries as well as on the main stream. It was to be hoped that that recommendation would be followed and that gauging-stations would be set up in various parts of the Severn catchment, in order that the relationships between the area and the nature of the catchment and the yearly off-flow and intensities of flood-discharges might be more completely investigated than had been hitherto possible in Great Britain. Such investigations would obviously depend for their completeness on self-recording rain-gauges placed at various points on the catchment. The Paper did not state what gauges of that type were situated above Bewdley, nor if steps were being taken to supplement them where necessary.

The application of the remark on p. 98 § that if, in the storm of the 31 May–1 June, 1924, $3\frac{1}{2}$ inches of rain had been discharged in 1 day, the peak of the flood would have been 180,000 cusecs, was a little difficult to follow. In the first place an average of $3\frac{1}{2}$ inches of rain did not fall on the catchment on that occasion, and in the second, even if it had, it would have been a physical impossibility for more than a portion of the total off-flow to have been discharged in 1 day. The period of concentration at Bewdley appeared to be somewhere about 36 hours, and obviously the total period taken by the rainfall to flow off was bound to be much more than that. The storm referred to was worth some study. It had been caused by a secondary depression of a thunderstorm type after a spell of hot weather. The distribution of the rainfall was shown in *Fig. 30* (p. 538). No part of the catchment seemed to have received more than 4 inches, but there were two small areas immediately to the north and south of it which had had more than 4 inches. On the catchment the average was just under 3 inches, or $\frac{1}{12}$ inch per hour during the period of concentration (if that were taken as 36 hours). With 100 per cent. run-off the maximum rate of discharge would have

§ *Ibid.*

Mr. Williams. been 87,700 cusecs. Actually it had been 14,600 cusecs, or 16·6 p
cent. of that figure. A total quantity of 11,360 million cubic fe
was represented by 3 inches of rain, and according to the hydrogra
the run-off was 5,580 million cubic feet, which agreed with t
Authors' figure of 48 per cent. of the whole fall. The small percenta
was probably due to the fact that the storm was immediate
preceded and followed by spells of hot dry weather. The hydrogra
indicated, in fact, just what might be expected to occur in th
storm. The rain had fallen in a period considerably shorter tha
the period of concentration and had been heavier on the lower par

Fig. 30.

Distribution of Rainfall on Severn Catchment above Bewdley
in Storm of 31 May–1 June, 1924.

of the catchment than on the higher. Part of the flood-water fro
the lower parts had had time to run off before that from the high
parts had arrived, and as the increasing tributary-area had bee
compensated for by a diminishing rate of rainfall, the dischar
had remained constant for a considerable time.

In February, 1928, the opposite conditions had occurred. Th
rainfall on the mountains on the periphery of the catchment had bee
very heavy and continuous, whilst on the low lands it had been litt
above normal. Consequently the flood had depended mainly on t
off-flow from the high land, and when that had ceased the rate
discharge in the river had fallen abruptly.

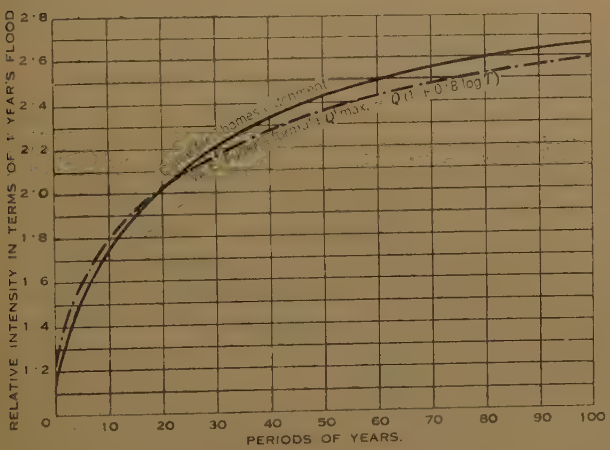
The peak of the probable maximum annual flood at Bewdl
seemed to be about 9,500 cusecs. The information in the Pap
would appear to show that the peak of the probable 15 years' flo

was 18,000 cusecs. The two figures were in the same ratio as he had found to apply to the Thames catchment. *Fig. 31* showed graphically the intensity of probable floods on that catchment in relationship to the periods between them, and also showed the curve for W. E. Fuller's formula, which was $Q_{max} = Q\,(1 + 0{\cdot}8 \log T)$. Mr. Williams

If the Thames curve were taken to apply to the Severn the probable 50 years' flood at Bewdley would be 22,500 cusecs and the 100 years' flood 25,300 cusecs.

He had been able to collect a good deal of information regarding off-flow/rainfall ratios in different parts of the world; *Fig. 32* (p. 540)

Fig. 31.



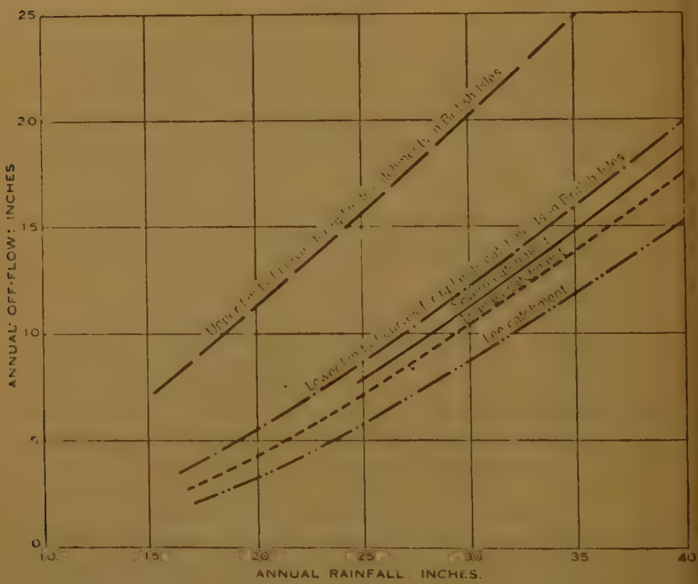
showed the limits within which the curves for annual off-flows for uplands catchments in the British Isles usually fell, as well as curves for the Thames, Lee, and Severn at Bewdley, the latter plotted from the data in *Fig. 17* (p. 107 §). As might be expected, it was below the lower curve for the uplands catchments, and above the Thames curve.

It would have been of interest to have had a diagram showing a longitudinal section of the river from its source. It had been held by investigators that the profile of a river became parabolic, and that when it broke through one or more well-defined ridges it formed a series of parabolas; the Severn had cut its way through the Wenlock-edge–Wrekin ridge formed by the band of Lower Silurian rocks, which cropped out at that point next to the New Red

§ *Ibid.*

Mr. Williams. Sandstone strata forming the floor of the wide valley extendi from Shrewsbury to the Mersey. A very large lake with a top-wat area of over 50 square miles could be constructed by building dam there, impounding water up to a level of 200·00 O.D. Th lake clearly did not come within the bounds of practical politi but a comparatively low dam would form a reservoir with a hig flood level of 173·00 O.D. and 15 miles long, with a maximum wat spread of over 16 square miles and a capacity of 6,000 million cu feet. If the normal top-water level were maintained at 168·00 O.

Fig. 32.



about 4,000 million cubic feet of storage could be utilized for strea regulation, which would enable a minimum flow of 700 cusecs to ensured below the dam. From the information supplied in the Pap it appeared that the minimum flow at Bewdley had been 300 cuse in 1922–23 (stated to be an average year), and 260 cusecs in 1929– and in 1933–34. If the portion of the reservoir above 168·00 O. were retained for flood-control, by suitable operation of the contr sluices the peak discharge of the 100 years' flood could be reduc by 33 per cent., and the 15 years' flood could be practically eliminat The effect of the reservoir would naturally grow progressively l towards the mouth of the river, and for complete control it would ne to be supplemented by other reservoirs, but the benefits obtain

from its construction would be substantial. He was not in a position to express an opinion as to how far those benefits would be worth the cost entailed. The most important items would be the acquisition of the land and buildings that would be submerged, and the diversion of several miles of the joint line of the L. M. & S. and G. W. Railways. Mr. Williams.

Mr. H. F. Wilmot observed that the extraordinary accuracy obtained from the three methods of gauge-reading over a period of 3 months could naturally only be achieved when spread over a long period of time, and then only for a steady-flowing river whose daily discharge showed no sudden peaks. Even in that case, however, there was no guarantee, either mathematically or practically, that the law of averages was operating evenly when only one reading of the gauge per day was taken, and therefore a slight lack of certainty was possible with regard to results deduced from such readings. The gauge was presumably placed within a stilling device to enable accurate water-levels to be read. Although probably wholly unnecessary, it would be of interest to have one gauge on each side of the river to check whether the pulsations of a river in flood gave irregularities on either side, which, read on a single gauge, would give either too high or too low a reading according to the time of taking it. Comparative charts from automatic records would be of interest but the duplication of apparatus would be too costly. Mr. Wilmot.

With regard to the calibration of current-meters, it would be interesting to know over what width of channel there was absence of interference from the sides; for example, a width of 2 feet was obviously inadequate. That would depend partly on the speed; but at any rate for high speeds 4 feet 6 inches seemed somewhat on the small side; any resulting inaccuracy would probably be small, and would give too low a rating.

For the calculation of the hydraulic radius in the case of a river-section where there was berm-flow, it had always been more practical to regard the flow as of two distinct types, and therefore the cross-section as composed of two different types, the main normal section, and the berms, and to calculate the value of each separately, as suggested by Mr. H. J. F. Gourley on p. 150 §. That appeared to give satisfactory results.

The Authors, in reply, observed that a number of interesting questions had been raised which could not be answered without considerably more investigation, and they commended those questions to future investigators. The Authors.

The decision to adopt a year commencing on the 1st October had been taken after a detailed consideration of the available information

§ *Ibid.*

The Authors. relating to British conditions, and it was satisfactory to note that t Inland Water Survey had adopted the same year. The fact th much useful information could be obtained from a close analysis o smaller catchment was not denied, but it had to be remembered tl the main object of the research was the measurement of the dischar of the river.

On the basis of Mr. Bransby Williams's diagram for the intens of probable floods (*Fig. 31*, p. 539), the flood of 1886, which w estimated to have reached a peak of 32,000 cusecs, at Bewdley w bound to be reckoned as very exceptional. It showed, howev that even the 100-year flood might be exceeded at any time. T special characteristics of the longitudinal section could not be broug out on a diagram of reasonable size, but the Authors would dra attention to the section printed on p. 21 of the Royal Geographi Society Report.[1]

Some experiments already published [2] showed that the rati established in the Imperial College channel only differed from that t the William Froude tank (30 feet wide by 12 feet deep) by less th ½ per cent. The commonest source of error in rating current-mete in a small channel was due to wave-action when the speed of towi was approximately equal to the velocity of a wave in the chann The width was also of importance, but recent investigations [3] Germany had shown that for a width of 1·5 metre (4·9 feet) the effe was very slight, and for widths in excess of 2·5 metres (8·2 feet) effect could be detected.

The Authors were indebted to Mr. Barnes for pointing out t discrepancies between the upper portions of the scale at the gaugin section. That observation showed the importance of freque checking of gauges founded on material other than solid rock. T lowest portion of the scale was fixed to a steel channel driven do to the rock, but the other portions were founded on concrete bloc sunk in the bank, and had evidently settled since the last observatio were made nearly 4 years ago. For that reason it was not consider that the stage–discharge relations in the Paper were likely to affected, but the error of the scale would need to be corrected considered in any future work. As the daily discharges were bas on the Bewdley scale they would not be affected.

The fundamental difference between current-meter discharg

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[1] The Investigation of Rivers, Final Report. Royal Geographical Socie 1916.

[2] M. A. Hogan, "River Gauging." Department of Scientific and Indust Research, London. H.M. Stationery Office, 1925.

[3] W. Henn, "Grundlagen des Wassermessung mit dem hydrometrisc Flügel." *V.D.I.-Forschungsheft* No. 385, 1937.

measurement and slope-discharge calculations was that in the former The Authors. the observations were confined to one cross-section and in the latter a long stretch of river had to be used. Uniformity of cross-section and straightness of the channel were desirable in current-meter work to avoid errors due to oblique velocities, but were not essential. In slope-measurement a length sufficient to yield a perceptible slope had to be taken, and it was assumed that the area, velocity, and wetted perimeter remained the same throughout. Variations in cross-section caused variations in slope and loss of energy due to the consequent acceleration and retardation of the flow. The common slope-discharge formulas took into account the roughness of the bed and sides, but in applying any slope-formula to a natural channel the variations in cross-section should also be allowed for.

During the dry summer of 1921 the Authors had taken many cross-sections of the river adjoining the gauging-section, and had concluded that the variations at low to medium flows were such as to cause an energy-loss comparable with that due to the roughness of the bed and sides. Since the two sources of loss of energy varied with the velocity there would be a relation between the slope and the discharge for any one stretch of river, but until some method for discriminating between the two sources of loss was discovered it would be impracticable to apply that relation to any other stretch of river.

Mr. Barnes had measured the surface-slope over a length of approximately 1,000 feet close to the left bank where there were over-hanging trees. It was interesting to speculate whether similar results would have been obtained on the right bank, because the line of maximum depth and discharge was closer to the left bank. The slopes varied from 32 to 78 parts in 1,000,000, but taking into account the nature of the measurements there was a possible variation of ± 5 parts in those results.

No doubt when Mr. Barnes published his proposed log-log formula full details would be given of the data on which it was based, but at first sight the influence of viscosity appeared to be over-emphasized. As was well known, the influence of viscosity in pipe-flow varied inversely with the roughness, and in a rough pipe the flow was independent of viscosity. The effect of viscosity was shown up by changes in the flow consequent on changes in temperature, and in some experiments [1] on very smooth large concrete pipes, where the influence of viscosity should have been most marked, Mr. Barnes had found the maximum variation due to a 19° F. change in temperature

[1] *Discussion on* "The Law of Flow through Disc-Orifices in Pipe-Lines," *and* "The Discharge of Pipes lined with Concrete or Bitumen," by A. A. Barnes. Trans. Inst. W.E., vol. xxxviii (1933), p. 186.

The Authors. to be less than 4 per cent. From studies of curves of vertic velocity distribution in the Severn, it seemed that the surface had be considered as approaching the fully rough condition, and th consequently there should be little or no viscosity-effect. With variation of 15° F. about the mean temperature the formula gav variation of $\pm$4·5 per cent. in the discharge; that was to say, flow for a given gauge-reading should be 9 per cent. greater in sumr than in winter. The discharge-measurements taken at vario times throughout the year afforded no evidence of a systema variation of that nature. It would be interesting to know t reasons which led Mr. Barnes to assume that the resistance of t channel at Bewdley would be the same as that for the Mucomir c A comparison between the calculated discharges and the stag discharge curve based on current-meter measurements appeared indicate a systematic divergence as the depth increased.

Gauge-reading: feet.	Discharge: cusecs.		Difference: per cent.
	Measured.	Calculated.	
5·40	380	386	1·6
5·63	500	508	1·6
5·69	525	542	3·2
6·30	870	913	4·9
6·42	940	979	4·1

It should be noted, however, that the divergence between t calculated and measured values was of the same order as t estimated error in the slope-observations. An error of $\frac{1}{16}$ inch slope corresponded to about $\pm$4 per cent. in the calculated discha at 6·42 feet gauge.

Mr. Griffith's difficulty with *Fig. 4* (p. 91 §) was to be attribut to the small scale of the diagram; the actual figures were as follow

Gauge: feet.	Cross-sectional area: square feet.
15·0	2,060
16·0	2,230
17·0	2,410
18·0	2,600
19·0	2,820

It would be seen from those figures that there was, in fact, a v

§ *Ibid.*

definite increase in width as the gauge-reading increased. A similar difficulty affected Mr. Griffith's determination of the gauge-reading for zero velocity, because for $d = 5{\cdot}25$, the velocity was approximately 0·5 foot per second, with a discharge of 305 cusecs. The Authors.

Both Mr. Griffith and Mr. Lacey referred to the use of the single surface-floats. In a straight drainage-channel or canal it might be possible to locate the position of maximum surface-velocity with some degree of accuracy, but the Authors' experience on the Severn was that the maximum surface-velocity altered its position with the stage of the river. Also, even if the position of the maximum velocity were known, there remained the difficulty of ensuring that the float entered the water there. On re-examining the surface-float results it was found that the value of the maximum surface-velocity varied irregularly, and it was not possible to connect those variations with the depth as had been done in *Fig. 19* (p. 111 §) for the average float-velocity. Taking the average of the group of about ten floats run close to the point of maximum surface-velocity in seventeen discharge-measurements, it was found that the average coefficient to be applied to the float-velocity to obtain the average velocity for the cross-section, and thus the discharge, was 0·66. The scatter of individual observations ranged from 0·52 to 0·84 with a single float, but if a number of floats were run and the average taken the range of coefficient was from 0·55 to 0·76. Thus the results for a single float would give the discharge within ±25 per cent., and for a group of floats within ±16 per cent. That error was considerably greater than the ±4 per cent. obtained when from thirty to sixty floats were distributed across the width and an average coefficient varying with the depth was taken as mentioned on p. 110 §.

Paper No. 5069.[1]

"The Open-Frame Girder."

By Gerald Salmon Gough, M.A., Assoc. M. Inst. C.E.

Correspondence.

Mr. E. H. Bateman observed that the Author had presented an interesting method for obtaining approximate solutions for the problem of the open-frame girder with parallel chords of uniform stiffness. Approximations of that type were, however, now unnecessary, since Mr. Bateman.

§ *Ibid.*
[1] p. 247, *ante*. (June, 1937.)

Mr. Bateman. the exact solutions of the elastic equations could be obtained by t method of remainder distribution which he had recently introduc for solving simultaneous equations of the type which arose in t analysis of that problem.[1] The exact solutions could be obtain just as easily as the Author's approximations, and moreover Mr. Bat man's method enabled variations in the stiffnesses of the chor sections to be included in the calculation just as simply as variatio in the stiffnesses of the web-members.

Following the method of analysis developed in his own Paper the equations connecting the moments M_1, M_2, etc., at the left-ha ends of the chord-sections were easily seen to be of the form

$$0 = -k_{n-1} \cdot M_{n-1} + (k_{n-1} + 6l_n + k_n) \cdot M_n - k_n \cdot M_{n+1} + \tfrac{1}{2} \cdot k_{n-1} \cdot S_{n-1} \cdot L_{n-1} - \tfrac{1}{2}(3l_n + k_n) \cdot S_n \cdot L$$

where L_1, L_2, . . . denoted the lengths of the first, second . . . chord-sections;

,, l_1, l_2, . . . denoted the values of L/EI for the chord-sections

,, k_0, k_1, k_2, . . . denoted the values of L/EI for the vertic sections;

,, S_1, S_2, . . . denoted the shears in the first, second . . . panels.

That equation, which was analogous to Clapeyron's equation three moments for a continuous beam, might for convenience i reference (and in default of a prior claim to title) be denoted Bateman equation of three moments for an open-frame girder, in which th upper and lower chords were parallel, and of which correspondin sections of the upper and lower chords had the same stiffness The equation could be derived by the method of analysis used by th Author, or by any other method which commended itself to thos who did not share Mr. Bateman's opinion of the advantages of analysi by the least-work system.

In the problem of the seven-panel girder, for which the Author gav approximate solutions by alternative methods, together with th results of an exact solution (the details of which were not given), th coefficients in the three-moment equations had the followin values :—

$L_1 = L_2 = L_3 = \ldots = L$, and the ratios of $l_1, l_2, l_3, \ldots : k_0, k_1, k$ . . ., might be written 1, 1, . . . 1 : 1, 1, 1·2, 1·6, 1·6, 1·2, 1, 1

[1] E. H. Bateman, "The Stress Analysis of Continuous Frames." Journ I. Struct. E., vol. xiv (1936), pp. 398 and 471.

—— "Remainder Distribution in the Analysis of Indeterminate Structures. *Engineering*, vol. xcliii (1937), p. 571.

[2] "The Open-Frame Girder." Journal Inst. C.E., vol. 1 (1935–36), p. 6 (November, 1935).

With unit loads at the second, third and fourth panel-points, which was the case described by the Author as "half-span loaded," the shears had the following values :— Mr. Bateman.

$$S_1 = 15/7,\ S_2 = 8/7,\ S_3 = 1/7,\ S_4 = S_5 = S_6 = S_7 = -6/7.$$

Entering those values in the three-moment equation, seven equations were available for the determination of seven unknown moments, thus

$$0 = 8M_1 - M_2 - 4{\cdot}286L$$
$$0 = -M_1 + 8{\cdot}2M_2 - 1{\cdot}2M_3 - 1{\cdot}329L$$
$$\ldots \text{etc.}$$

The coefficients in those equations, together with the solution of the equations by remainder distribution, were set out in the following Table :—

M_1	M_2	M_3	M_4	M_5	M_6	M_7	$L/1000$	R_1	R_2	R_3
8	−1						−4,286	−486	−26	−4
−1	8·2	−1·2					−1,329	−129	−13	—
	−1·2	8·8	−1·6				357	−3	−3	−2
		−1·6	9·2	−1·6			2,086	646	18	4
			−1·8	8·8	−1·2		1,114	−86	26	2
				−1·2	8·2	−1	1,200	0	−10	−2
					−1	8	1,286	−114	−34	−3
500	200	−50	−200	−200	−200	−200	First partial solution.			
60	20	−10	−70	—	—	10	Second ,, ,,			
3	2	—	−2	−3	1	4	Third ,, ,,			
563	222	−60	−272	−203	−199	−186	Final solution.			

In the Table the coefficients of $L/1{,}000$ had been used instead of the coefficients of L, in order to avoid the tabulation of fractional solutions and remainders. The partial solutions were given to one significant figure only, and they were obtained as follows: the first partial solution for M_1, given as 500, was obtained by dividing −4,286 by 8, the coefficient of M_1 in the first equation, and changing the sign; that value of 500 was then used in the second equation together with the initial remainder of −1,329, thus :—

$$\{500 \times (-1) - 1{,}329\}/8{\cdot}2,$$

which gave 200 (to one significant figure); when the first series of partial solutions had been estimated in that way, those solutions were used in the original equations to calculate the remainders tabulated under R_1; those remainders were then used to estimate a second series of partial solutions; the process was repeated until the initial remainders had been sufficiently distributed, and the final solutions

Mr. Bateman. were found by summing the corresponding partial solutions. Th abbreviated and improved form of the remainder-distributi method had first been published by Mr. Bateman in a Paper read the Nottingham meeting of the British Association on the 2 September, 1937.

Thus the values of the seven moments M_L at the left-hand ends the chord-sections were :—

$0{\cdot}563L$; $0{\cdot}222L$; $-0{\cdot}060L$; $-0{\cdot}272L$; $-0{\cdot}203L$; $-0{\cdot}199L$; $-0{\cdot}186L$;

the values of $SL/2 = M_L - M_R$ for each section were :—

$1{\cdot}071L$; $0{\cdot}571L$; $0{\cdot}071L$; $-0{\cdot}429L$; $-0{\cdot}429L$; $-0{\cdot}429L$; $-0{\cdot}429L$;

and the values of M_R, the moments at the right-hand ends of t chord-sections, were :—

$-0{\cdot}509L$; $-0{\cdot}349L$; $-0{\cdot}131L$; $-0{\cdot}157L$; $-0{\cdot}226L$; $-0{\cdot}230L$; $-0{\cdot}243L$.

The Author. The AUTHOR, in reply, observed that Mr. Bateman expressed th opinion that a method of approximating to the stress-distribution an open-frame girder was unnecessary, as the equations could easi be solved by his method of remainder distribution. To the Auth that was not obvious. For any structure which was not simply st there was a difference between the requirements of the designer a of the checker. The former had to arrive at suitable cross-sectio from the loads and the working stresses. At present that could on be achieved by a process of trial and error, for which, it might supposed, quite rough preliminary calculations might be ve valuable. The checker, on the other hand, knowing his cros sections, could and should be precise. For him the full set simultaneous equations was suitable, and they could be solved Mr. Bateman's or any other method. It seemed that there mig well be room for distinct approximate and precision methoc Further, the Author would not presume to express an opinion as which was the better or quicker of two methods, one of which knew and the other of which he would have to learn. In any ca he preferred to have more than one line of approach available.

The exact solution in the Paper was obtained by direct solution the simultaneous equations, a simple process which did not appear require comment. The equation obtained by Mr. Bateman did appear to be of particular importance.

LATE CORRESPONDENCE
ON PAPER PUBLISHED IN JUNE 1936 JOURNAL

Paper No. 4989.[1]

"Corrosion of Iron and Steel."

By Sir Robert Abbott Hadfield, Bart., D.Sc., D.Met., F.R.S., M. Inst. C.E., and Sidney Arthur Main, B.Sc.

Correspondence.

Dr. Carl Benedicks, of Stockholm, thought that, in the very **Dr. Benedicks.** arge amount of valuable information presented in the Paper, a triking point was the regularity of the results obtained, as shown in *Figs. 4, 5* and *6* (pp. 25 § *et seq.*), between the observations under erial, half-tide, and total-immersion conditions obtained at Halifax, uckland, Plymouth and Colombo. That induced him to attempt o explain the climatic influences in more detail.

Meteorological Data.

Dr. A. Ångström had provided the following data showing the ange of temperature at each place, the maximum and minimum alues of the relative humidity, with their averages, and the rainfall,

[1] Journal Inst. C.E., vol. 3 (1935–36) p. 3. (June, 1936).
§ Page numbers so marked refer to the Paper.—Acting Sec. Inst. C.E.

Dr Benedicks. the localities being arranged according to the average annual a temperature :—

Location.	Average annual tempera-ture: °C.	Range of tempera-ture, $T_{max}-T_{min}$: °C.	Maximum relative humidity: per cent.	Minimum relative humidity: per cent.	Average relative humidity: per cent.	Yearly rainfall millimetr
Halifax	6·2	12·6	86	77	82	1390
Plymouth	10·5	9·7	88	75	82	1080
Auckland	15·2	8·5	82	73	78	1110
Colombo	26·8	1·7	81	73	77	2240

Although the above temperatures were air-temperatures, th were bound sensibly to apply also for sea-water.

General Wastage under Total-Immersion Conditions.

It was natural to expect the general wastage to increase co siderably with temperature under total-immersion condition that was to say, in the order in the above Table. The action of t local elements, now almost universally accepted as the essent factor in corrosion, was actually known to increase with temperatu (the electrical conductivity increased considerably with temperatur The average-wastage figures observed for ordinary steels and ir were, however :—

Halifax	0·51 millimetre.
Plymouth	0·48 ,,
Auckland	0·43 ,,
Colombo	0·49 ,,

That was shown by *Figs. 4* (p. 25 §) and by *Figs. 12* (a) and *13* (Thus, there was actually no increase in wastage at all (or even slight decrease) with increasing temperature. On account of t increase in conductivity of the electrolytes on temperature increa (2 per cent. per degree), 0·51 millimetre at Halifax would correspo to 0·72 millimetre at Colombo. The difference considerably exceed the possible experimental errors.

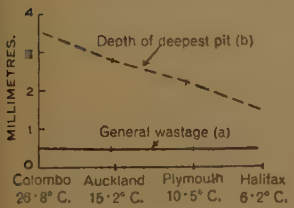
That remarkable fact proved that there was necessarily so foreign factor, besides corrosion itself, which influenced the gene wastage of the totally-immersed specimens. It was not possi to state definitely the character of that foreign factor, but he thou it likely that the cause of the constant wastage was to be found

§ *Ibid.*

the formation of an essentially protecting or inhibiting layer on the surface of the specimens immersed. **Dr. Benedicks.**

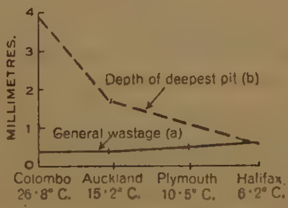
It was well known that sea-water contained considerable constant amounts of salts of magnesium (about 0·4 per cent. of $MgCl_2$) and calcium (about 0·2 per cent. of $CaSO_4$). During corrosion, caused by the action of the local elements, hydroxyl ions were formed at cathodic portions, making the liquid alkaline there, and precipitating hydroxide of magnesium (and possibly also of calcium) besides hydroxide of iron. The hydroxides would soon change into carbonates (whereby calcium carbonate might also be precipitated), and they would form a coating, protecting the metal to some extent

Fig. 12.



STEEL.

Fig. 13.



IRON.

TOTAL-IMMERSION CONDITIONS.

against continued corrosion. He had had the opportunity of studying in detail several cases of corrosion of Swedish sulphite-pulp boilers, and he had been able to establish that the possibility of using iron or mild steel plate in the boilers in presence of so reactive a substance as sulphite lye was entirely due to the fact that a protecting coating, essentially consisting of calcium carbonate, was successively formed. If that coating were allowed to form for too long a time without being removed, it would finally become so thick that, although adhering strongly, cracks would appear, permitting the coating to scale off locally. At such points extremely strong corrosion occurred until the pressure of the hydrogen formed on corrosion counteracted further attack.

Thus the formation of a thin and partially-protective coating in ocean water seemed to some extent to be supported by actual experience. The temperature would necessarily influence the speed with which that coating was formed, the coating forming more rapidly the higher the temperature, but the actual attack of the metal would be about the same, irrespective of the temperature, provided that the water had practically the same composition as in the open ocean.

Dr. Benedicks. The assumption that such action occurred would explain th interesting uniformity of wastage independent of temperature. H could see no other probable explanation[1].

Pitting under Total-Immersion Conditions.

He would next consider the fact that a protective coating such a that referred to above would always present local defects. At a poi thus unprotected, considerable local corrosion would set in; th renewing of the protecting coating there seemed rather improbabl Such local corrosion, or pitting, would necessarily be expected t increase strongly with temperature. *Figs. 5* (p. 26 §), *12* (b) an *13* (b), for ordinary steels as well as for rolled irons, showed tha very marked increase of pitting with rise in temperature occurre under total-immersion conditions.[2] Further, the sequence of th localities actually plotted in *Figs. 5* was found to coincide with th sequence of the temperatures of the different localities.

Pitting at Half-Tide: Iron Compared with Steel.

It was now necessary to consider the corrosion in the half-tid experiments.

If it were supposed that the specimens, when in air, dried imme diately and completely, the general wastage would be just one-hal that under total-immersion conditions, but such a supposition wa very far from being true. On account of the presence of salts i solution the surface of the specimens would, even when not immersed locally be in contact with a salt solution, and would thus locall corrode, or pit. That pitting-corrosion would be much stronger tha for the immersed specimens, on account of the free access of oxyge and probably also on account of a higher salt-concentration.

Such pitting was clearly dependent on two factors:

(i) As remarked above, the speed of pitting would increase wit a rise in temperature; that would tend to render the pitting maximum at Colombo.

---

[1] It might be assumed that the uniformity of corrosion of totally-immerse specimens was due to the temperature of the sea being almost the same at th localities discussed, but in view of the fact that at Halifax the winter tempera ture of the sea was about 0° C. and the summer temperature about 12° C while at Colombo the temperature of the sea varied between 25° C. and 30° C during the whole year, that explanation was excluded. The sea temperatur in fact, was one of the main factors determining the air temperature.

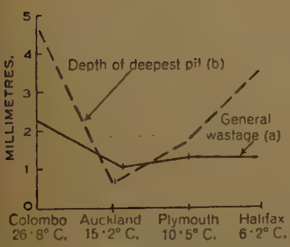
[2] The increase appeared still more regular if the localities at which test had been made were plotted at distances apart proportional to the actu temperatures rather than at equal distances apart.

(ii) The amount of solution occurring causing corrosion would be nuch more considerable at a place of low temperature, especially f combined with a high relative humidity, than at a place of high emperature, especially if combined with a lower humidity. Conequently, that factor would cause the pitting to be a maximum t Halifax (average temperature 6·2° C., average humidity 82 per ent.), and much more pronounced than at Colombo (average emperature 26·8° C., average humidity 77 per cent.). The great ange of temperature at Halifax (12·6° C.) would also contribute to he pitting. Dr. Benedicks.

With regard to the observations of pitting for ordinary steels, s shown by *Figs. 5* (upper part), and *14* (b), it would be seen that he maximum theoretically expected for Colombo was very prominent, as well as the maximum for Halifax. For the rolled irons, s shown by *Figs. 5* (lower part) and *15*, the occurrence of the above wo maxima was also clear.

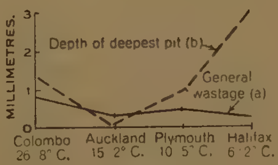
It was noticeable that the pitting of steel was in every case considerably stronger than that of iron; that difference was especially narked at the highest temperature (Colombo). The same difference

Fig. 14.



STEEL.

Fig. 15.



IRON.

HALF-TIDE CONDITIONS.

etween steel and iron was to be seen for the pitting under totalmmersion conditions (*Figs. 12* (b) and *13* (b); steel was much more orroded than iron at Halifax, Plymouth, and Auckland, whilst at Colombo practically the same value was obtained. Further, the otal wastage of steel under half-tide conditions was much higher han that of iron (*Figs. 14* (a) and *15* (a)). Under aerial conditions o definite statement could be made (*Figs. 16* (a) and *17* (a)).

The fact that the steel specimens generally were more corroded han those of iron might be explained by the assumption that the urfaces of the steel specimens were less smooth than those of the

Dr. Benedicks. iron specimens ; in view of the great care given to the experimen preparations, however, that seemed very improbable. Steel, the fore, appeared to be more corrodible than iron. From the sta point of the theory of local elements that was precisely what was be expected. In both cases the carbon dissolved in the ferrite w bound to be sensibly the same (about 0·03 per cent. of carbon d solved). The difference, therefore, would be solely that the st contained much more of the free-phase cementite, possessing "nobler" electrolytic potential than ferrite, and consequen causing a more effective action of local elements in steel than in ir

At the start of half-tide corrosion, the difference between steel a iron would not be considerable, but after continued exposure t surface of the steel would grow more and more rough, whilst t surface of iron would be less altered. It would therefore be se that the stronger pitting of steel, as shown by *Figs. 14* (b) and *15* (and also its higher general wastage (*Figs. 14* (a) and *15* (a)), seemed be theoretically satisfactorily explained. It might be added that sulphite lye, steel was attacked considerably more than iron.

General Wastage under Half-Tide Conditions.

In the half-tide tests the corrosion would take place alm exclusively at the points where liquid was retained ; that was say, it would occur mainly at a rate proportional to pitting. Henc the general wastage at half-tide could be expected to be approximate proportional to the pitting.

If *Figs. 4* (upper part) or *Fig. 14* [1] (a) were compared with t curve for pitting (*Fig. 14* (b)), it would be seen that that co clusion was not well verified, in so far as the wastage at Halif was much less than would be supposed from the pitting there.

In his opinion, the reason of that would be found in the atmosphe conditions at Halifax. On account of the great amount of ra (1,390 millimetres) falling there during the year, and also due fogs, which exerted a moistening and washing influence, the ma portions of the surface of the specimens were bound to be w purified from salt, the effect being to reduce the general wast considerably ; that would not necessarily reduce the pitting effectively, as salt might persist in the pits. It might be object that at Colombo the amount of rain in a year was much grea (2,240 millimetres) than elsewhere ; that rain fell, however, duri

---

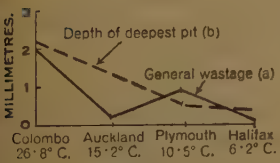
[1] *Figs. 4* would have to be redrawn so as to give the same sequence localities as *Figs. 5* ; that was to say, the sequence of temperature. In orde avoid that, *Fig. 14* had been drawn.

much shorter period, so that the effective washing action might Dr. Benedicks.
e much less. Taking that fact into consideration, the difference etween the general wastage and pitting at half-tide was not ifficult to understand. It might be added that a certain protecting oating might also be formed during half-tide tests, but it was bound ɔ be much less protective than if formed quietly under total-nmersion conditions.

eneral Wastage and Pitting on Aerial Tests.

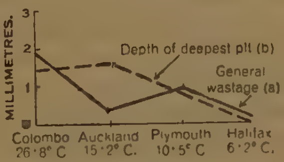
The pitting in aerial tests increased fairly regularly with increasing emperature for both steel and iron (*Figs. 5, 16* (b), and *17* (b)). That eemed quite natural in view of the rate of attack increasing with emperature, but, on the other hand, it proved that, even at the ottest locality, liquid remained on the surface of a specimen. That eemed to him to indicate the presence of a certain amount of salt, vhich was natural at any locality situated in the vicinity of the cean, as were the four sites selected. At a place like Delhi, how-

Fig. 16.



STEEL.

Fig. 17.



IRON.

AERIAL CONDITIONS.

ver, situated far from the coast and from salt-deserts, it was very mprobable that liquid would remain on the surface of an iron object ike the well-known pillar, and thus the general wastage would be xpected to be very small, as compared with that of Colombo. A urther point was that the rain falling in the south of the Himalayas robably contained some nitric acid, which tended to form an xidized preserving coating on an iron object.

The general wastage, on the whole, could be reasonably compared vith the pitting, as shown by *Figs. 16* (a) and *17* (a) (compared with *'igs. 16* (b) and *17* (b)). The general wastage at Plymouth, however, vas evidently much greater than that at Auckland, which could not e expected from the temperatures. The fact that the relative umidity at Plymouth (82 per cent. average) was greater than at Auckland (78 per cent. average) might appear to explain that

Dr. Benedicks. anomaly; further, it seemed natural to assume that the air at first place was more polluted by traces of sulphuric acid than at second, due to a greater industrial use of coal.

At Halifax the depth of pitting of iron was found to be z (*Fig. 17* (b)), whilst some pitting occurred in steel. That diff ence might be explained by the greater smoothness of the irons, already mentioned. Some further reason, however, seemed necess to explain the lack of pitting at Halifax. Such a reason might found in the frequent occurrence of rain and fog at Halifax, wh would effectively wash away the salt-particles existing everywh near the sea-coast. If the surface was smooth, no pitting wou therefore, arise.

The Influence of Scale.

As emphasized by Dr. U. R. Evans (p. 615‡), it had been found th mill-scale led to pitting, although on the average it had no appr ciable influence on the corrosion as a whole. It might be appropria however, to remember that the influence of scale was necessar two-fold. A mill-scale coating possessed electrolytically a mu "nobler" character than iron or steel; as long, therefore, as t scale-coating was a continuous one, it would preserve the mater against attack and would reduce the general wastage to an extreme low figure. As soon as cracks became appreciable, however, stro pitting would result, since the action of local elements between scale-free portion and the surrounding scale-coating was necessar a very strong one. Scale would, therefore, at first give a ve low corrosion-loss, but later a very high one. Those influenc might compensate each other, so that no appreciable influer resulted.

The "noble" character of the scale was probably connected with t fact mentioned by Dr. H. B. Footner (p. 617‡), that paint, whi adhered satisfactorily to a free iron surface, possessed a poor adhesi to surfaces covered with scale; that point had generally been ov looked. Thus, the well-known ineffectiveness of painting on a sca bearing surface was not essentially due to a loosening of the sca but to a loosening from the scale. That was a very interesti observation, which clearly needed some explanation. At prese very little was known regarding the adhesion of a substance another substance, or the wetting between a liquid and a sol It was almost certainly the case, however, that the adhesion wetting was good in any case where a chemical reaction took pla

For example, he had discovered that, in vacuo, pure molten i

‡ Journal Inst. C.E., vol. 3 (1935–36) (October, 1936).

did not wet an aluminium-oxide crucible, whilst molten steel did. Dr. Benedicks. The reason was bound to lie in the fact that the carbon of the steel reacted with the crucible walls, at least to some extent, whilst between carbon-free iron and the crucible walls no appreciable reaction took place. If it were assumed that the more intense the reaction taking place, the stronger was the adhesion, it followed that the linseed oil of the paint, containing acids (in the form of isolinoleic acid) which partially attacked the iron, caused a strong adhesion. The iron oxide ($FeO + Fe_3O_4$), however, having a more "noble" character, was probably considerably less attacked, so that the adhesion might be much lower. If an addition could be made to the oil so that the iron oxide was slightly attacked, a good adhesion would probably result.

Corrosion-Fatigue.

Mention was made on p. 10 § of what was known as "corrosion-fatigue." There was no doubt that a corrosive liquid, which, as pointed out above, wetted the metal, might, if the metal were stressed, easily penetrate through grain-boundaries or small cracks, thus considerably reducing the strength of the material. He was convinced, however, that the true corrosive action of the liquid was unnecessary; if the surface of a stressed solid were surrounded by a liquid which merely wetted it (without appreciably attacking it) that was bound to be sufficient to cause a very considerable lowering of the strength of the material. A good example was given by glass strips, which had been scratched with a diamond under constant pressure; it was found that the breaking load of the strip, when the specimen was wetted with distilled water or other liquid, was diminished to less than 50 per cent. of the value obtained on a dry specimen (that was to say, with air as the outer medium in contact with the surface of the specimen).

Summary.

The preceding analysis of the data enabled certain conclusions to be drawn which might be summarized as follows:—

(1) Very approximately, the depth of pitting, in total-immersion as well as in aerial tests, increased roughly in proportion to the air temperature.

(2) The half-tide tests showed that on that influence of temperature on the rate of pitting, there was superimposed an opposite effect, which was weak at high, and strong at low, temperatures. That effect, no doubt, was the easy condensation of water at low tempera-

§ *Ibid.*

Dr. Benedicks. tures, which was naturally the primary cause of corrosion. On account of the presence of salt, the occurrence of much water, at low temperatures, caused a very strong pitting in half-tide tests.

(3) The most striking fact regarding general wastage under conditions of total immersion was that it did not increase with increasing temperature, as might be expected. The cause was doubtless that a protecting coating would rapidly be formed on iron or steel surfaces in sea-water, the formation of the coating being comparable with what was known regarding certain protecting coatings occurring in industry.

(4) The effect of humidity in accelerating corrosion might under certain circumstances be the opposite to that stated in (2) above. A large amount of rain and fog during a great part of the year was bound to have the effect of cleaning the surface of the specimens, so counteracting corrosion; that was especially true in the tests at Halifax. That cleaning action, however, scarcely appeared to remove the salt existing in the pittings formed.

(5) The fact that the aerial and half-tide wastage at Plymouth was more intense than that at Auckland might be due to the higher relative humidity at Plymouth, but might possibly also be due to a less pure atmosphere there.

(6) Carbon steel was generally more corroded than iron, which was what was to be expected.

(7) From the foregoing the essential character of the results observed on iron and carbon steel seemed to be fairly well explained by considering the known meteorological factors, on the theory of local elements. The only special assumption made was the formation of a protecting coating in sea water (Conclusion 3), and eventually of a less pure atmosphere in Plymouth (Conclusion 5). No individual differences in the action of the sea water, nor any other unknown factors, were necessary.

(8) An explanation was put forward for the reason why paint adhered better to iron than to iron oxide.

(9) It was pointed out that "corrosion-fatigue" was merely a special case of a general effect, implying that the strength of a material was effectively lowered by the presence of a wetting liquid medium surrounding the material.

The Authors. The AUTHORS, in reply, observed that a communication from such an eminent authority as Dr. Carl Benedicks would, they were sure, be appreciated and would be studied with considerable interest by those following the work of the Sea Action Committee. Of particular interest was the explanation afforded for the remarkable degree of uniformity of the wastage by corrosion of steel and iron totally immersed in the sea under the various climatic conditions, even

though, as Dr. Benedicks rightly pointed out, the temperature of the sea as well as that of the air varied to a great extent between the different stations. The Authors.

There were several factors connected with sea-water which might make its action different from that of simple salt solutions in the laboratory. Instances of the action of "inhibitors" in comparatively small proportions in modifying the attack of corrosive agents were many, whether through their chemical action or by the formation of protective films; the possible action in the latter way, which Dr. Benedicks suggested, of the magnesium and calcium salts which existed in sea-water, should therefore receive consideration, based as it was on their observed action in other directions. How far the formation of a film from those constituents might co-operate with, or be hindered by, the presence of molluscs was naturally a point to be taken into account.

A further interesting suggestion was with regard to the maxima of pitting action found under half-tide conditions, both in the hot climate at Colombo and the cold one at Halifax, and there seemed much to be said for the idea that greater condensation under the latter conditions could be equally effective in causing pitting as the higher temperature and generally drier condition of the steel in the former. In that connexion, the Authors had indicated a further factor as likely to have been operative in causing excessive pitting under half-tide conditions at Halifax, namely, the presence of oil for a period, an idea which seemed to be borne out by the results of the 10-year specimens, which did not show the same excessive pitting.

Regarding the relative corrosion of the rolled irons and the ordinary steels, such superiority as the former possessed was considerably modified when comparison was confined to specimens which had been exposed with their rolling-scale removed. Under such circumstances, whilst the irons were slightly but distinctly superior to the steels in general wastage, they were inferior in respect of pitting. Thus, referring to Tables IX to XII (pp. 73–76 §), whilst the general average of the wastage of the irons was 0·579 millimetre, that of steels A and C was 0·617 millimetre; the corresponding figures for pitting were 1·38 and 0·92 millimetre respectively.

The Authors were glad to have Dr. Benedicks' observations on the question of the Pillar at Delhi, and his further confirmation of their conclusion that its excellent state of preservation was to be attributed to favourable climatic conditions rather than to any special corrosion-resisting properties of the iron of which it was formed. Those engaged in the intensive study now proceeding on

§ *Ibid.*

The Authors. the protective influence of paints would no doubt also welcome the suggestions made by Dr. Benedicks regarding the factors affecting the adhesion of paint to a scale-bearing surface.

The causes of the peculiar behaviour known as "corrosion-fatigue" were still obscure and were for the present largely a matter of conjecture; whilst it was natural to suppose that corrosive action was bound to be at work, that, as Dr. Benedicks rightly pointed out, was by no means certain. The subject was a most interesting as well as important one, and as suggested, research in physical science in other directions might, well be brought to bear on the problem.

Paper No. 5084.

"Experiments on the Flow of Water in Pipe-Bends."

By Herbert Addison, M.Sc., Assoc. M. Inst. C.E.

(Ordered by the Council to be published in abstract form.)[1]

The experiments described in the Paper were begun in the hope of resolving some of the doubts that, in spite of the information recently published on the subject by The Institution,[2] still remained. The bends used were of square cross-section, of 6-centimetre (nominal) side, and 12-centimetre (nominal) mean radius, and they could be built up together with lengths of intervening 6-centimetre-square straight pipe into various systems of continuous and reversed curvature (*Figs. 1*, p. 562). The complete system was laid in a horizontal plane, and the mean water-velocity through it ranged between about 2 and 5 metres per second. In one set of experiments the bends and straight pipes were of brass, smooth-machined over the entire internal surface, and in the other they were of unmachined cast iron.

The various arrangements used are shown in Table I (p. 563); thus in series 3, ACDBF(LR), the water flowed in turn through inlet-bend A, two straight sections C and D, outlet-bend B, and outlet straight section F. The first bend is a left-hand bend and the second a right-hand, and this is indicated by the letters LR appearing in brackets at the end of the code expression of the arrangement of pipes. The mean values of the ratio $\frac{\text{apparent energy-loss}}{\text{velocity-head}}$, or the coefficient of loss, relate to mean velocities of from 3 to $3\frac{1}{2}$ metres per second, and were computed as follows:—At each of the measuring planes aa, cc, etc. (*Figs. 1*), there were five pressure-orifices spaced

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[1] The MS. and drawings can be seen in the Institution Library.—Acting Sec. Inst. C.E.

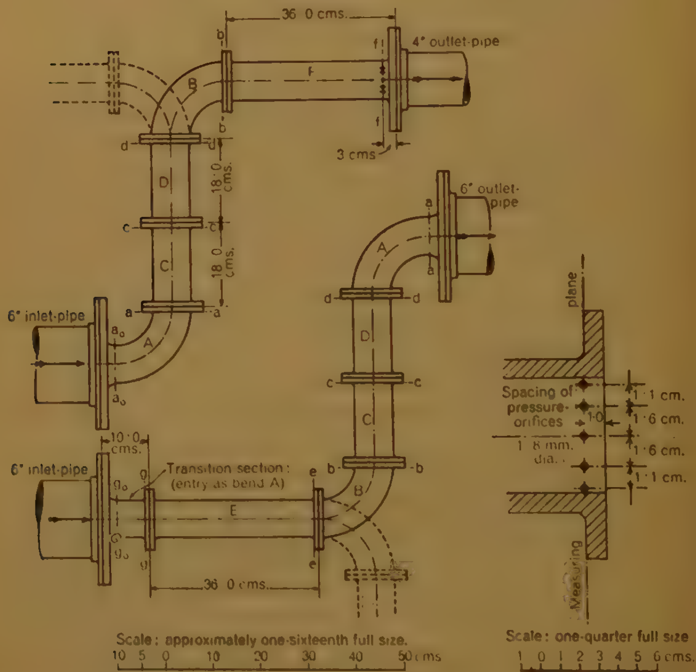
[2] A. H. Gibson, T. H. Aspey and Fred Tattersall, "Experiments on Siphon Spillways." Minutes of Proceedings Inst. C.E., vol. 231 (1930–31, Part 1), p. 203.

Powys Davies, "The Laws of Siphon Flow." Minutes of Proceedings Inst. C.E., vol. 235 (1932–33), p. 352.

Note.—The Paper on "The Flow of Water through Rectangular Pipe-bends," by Powys Davies and S. V. Puranik (Journal Inst. C.E., vol. 2 (1935–36), p. 83, February, 1936), had not been published when the Author's experiments were made.

as shown, and from the observed pressure-head at each orifice the mean pressure-head at the measuring plane was computed. The apparent energy-loss between any two measuring planes was taken as the difference between the mean heads at those planes. The last column of Table I was obtained by adding to the observed losses in the bends the observed loss in the intervening straight

Figs. 1.



pipes when these were tested as a complete straight system without bends.

From Table I it appears that :—

(1) rough cast-iron bends or systems usually have much greater coefficients of loss, in equivalent conditions, than smooth brass bends ;

(2) changing the " hand " of one bend of a system from right hand to left hand or *vice versa* has little definite effect on the overall loss in the system, though it may considerably influence the apparent loss in the individual bend ;

TABLE I.—ANALYSIS OF HEAD-LOSSES.

Reference number of series.		Arrangement.	Number of experiments utilized.	Mean values of the ratio $\frac{\text{apparent energy-loss}}{\text{velocity-head}}$. In inlet or upstream bend.	In outlet or downstream bend.	Total loss in system.	Sum of separate losses.
BRASS	12	ABF(LR)	3	—0·0	0·081	0·167	
	2	ACBF(LR)	4	—0·019	0·124	0·227	
	3	ACDBF(LR)	4	—0·026	0·142	0·277	
	4	ACDEBF(LR)	4	—0·029	0·122	0·381	0·372
	6	ABF(LL)	5	—0·067	0·091	0·131	
	7	ACBF(LL)	5	—0·055	0·116	0·208	
	8	ACDBF(LL)	5	—0·054	0·111	0·269	
	9	ACDEBF(LL)	5	—0·057	0·118	0·374	0·336
BRASS	13	EBA(LR)	3	0·150	0·186	0·437	
	14	EBCA(LR)	3	0·087	0·220	0·486	
	16	EBCDA(LR)	4	0·085	0·272	0·594	
	15	EBCDFA(LR)	4	0·081	0·327	0·759	0·642
	17	EBA(RR)	6	0·035	0·288	0·432	
	18	EBCA(RR)	6	0·088	0·271	0·489	
	19	EBCDA(RR)	6	0·085	0·317	0·605	
	20	EBCDFA(RR)	6	0·089	0·363	0·767	0·637
IRON	28	EBA(LR)	6	0·289	0·211	0·674	
	27	EBCA(LR)	6	0·169	0·219	0·706	
	26	EBCDA(LR)	6	0·170	0·348	0·854	
	25	EBCDFA(LR)	7	0·172	0·494	1·061	0·923
	21	EBA(RR)	6	0·139	0·269	0·581	
	22	EBCA(RR)	6	0·179	0·238	0·665	
	23	EBCDA(RR)	6	0·154	0·369	0·826	
	24	EBCDFA(RR)	6	0·162	0·493	1·041	0·830

(3) the coefficient of loss of a bend may be greatly influenced by its position in the system, and therefore it is not permissible to estimate the total loss in a system by adding to the losses in the bends the measured loss in the straight elements.

Examination of the effects of the joints in the system, and of the error involved in taking the mean pressure-head to represent the energy at each measuring plane, showed that these were normally too small to invalidate the above conclusions. The influence of the mean velocity in the system on the coefficient of loss was found to depend on the smoothness of the material and on the arrangement of the bends. The value of the exponent n in the expression

$$\text{head-loss in system} = \text{constant} \times (\text{mean velocity})^n$$

varied from 1·89 for arrangement EBCDFA(LR), brass, to 2·01 for arrangement EBA(LR), iron. When the system approximated to a long, smooth, straight pipe, therefore, the overall coefficient of loss diminished slightly as the mean velocity increased, but the coefficient of loss was independent of the mean velocity when bends predominated in the system, and the pipe-walls were rough.

After the experiments summarized in Table I were finished, a few further tests were made to find the effect of the outlet conditions on the performance of the downstream bend; thus, instead of allowing brass bend A, when forming part of the system GBCDEA(LR), to discharge into the 6-inch outlet-pipe, it was connected to an open-top tank. This, however, had no sensible effect on the coefficient of loss, which merely diminished from 0·327 to 0·311. On the other hand, when brass bend B was the downstream bend of the system ACDEB(LR), it had the following values of coefficient of loss: with 6-centimetre square section F downstream, 0·122; discharging directly into 6-inch outlet-pipe, 0·159; discharging directly into open-top tank, 0·093; with short flared section G downstream, and then discharging into open-top tank, 0·095. It appears, therefore, that the flared outlet of bend A, in which the section is increased from 6 centimetres square to 10 centimetres square in a length of 3·2 centimetres, although downstream of the measuring length, is responsible for more than half the loss in the bend.

Paper No. 5104.

"A Proposed Formula for use in Traffic Surveys, Particularly Applicable to the Construction of Branch-Line Railways in India and Similar Countries."

By Wilfred Eric Randall Gurney, M. Inst. C.E.

(*Ordered by the Council to be published in abstract form.*) [1]

In estimating the dividend prospects of a proposed railway, it is generally admitted that, after the completion of an engineering survey, all the factors required can be known with sufficient accuracy except the estimated gross earnings of the projected line, unless a railway is to be constructed for a definite purpose such as to serve a new harbour or a mining area, and in certain other cases. The majority of railways in India and in countries with similar development to India are, however, not constructed to serve an area with any particular industries or to shorten the route to some important centre, and thereby to enable the constructing company to compete more favourably with a rival railway system. They are constructed simply because the area they are to pass through is not served by any other nearby railway, and is sufficiently large and populous to make a branch-line construction through the area a reasonable proposition. In such cases it is found that there is considerable difficulty in estimating satisfactorily the gross earnings of the future line and, consequently, in deciding whether the project is a paying proposition or not.

It was recognized in India some 40 years ago that the dividend prospects of such lines depended more or less on the density of population of the country through which the projected railway was to pass, and, since then, it has been customary in India to take the density of population of the country traversed into consideration in making traffic surveys for new projects. Motor-cars and motor-buses have, however, in recent years reduced railway earnings considerably, and made it necessary to estimate the dividend prospects of a proposed railway with greater accuracy than formerly. Moreover, buses have increased the effective range of a railway

[1] The MS. and drawings can be seen in the Institution Library.—Acting Sec. Inst. C.E.

station, and roads running at right angles to existing railways and served by motor buses act more or less as feeder lines and bring in traffic. Parallel and nearby roads on the other hand take traffic away from them. Formulas which in the past gave sufficiently accurate results can no longer be relied on to do so.

The Paper describes an empirical formula for calculating the earnings of a projected railway, obtained after comparing the actual earnings of some forty representative stations on the Great Indian Peninsula Railway with the area of land served by each station; the population-density in the area; the average price of land; the average cost of labour; the mileage and direction of the roads in the area; the size of the largest town served and its distance from the station; the type of town served, and so on.

The figure for coaching earnings is obtained by taking the population of each town and village in the area served by a station and multiplying these figures by a number, which for towns of over 20,000 inhabitants would be 6, for towns between 10,000 and 20,000 inhabitants 5, for towns between 5,000 and 10,000 inhabitants 4, for towns between 2,000 and 5,000 inhabitants 3, and for villages of under 2,000 inhabitants 1. For all towns and villages within 1 mile of a metalled road but over a mile from the station the above figures should be doubled. The figures thus obtained are marked on a 1-inch map against the various villages and towns, and the area served by each station is sub-divided into sub-areas by drawing circles with a common centre at the site of the railway station or proposed station with radii of 1, 2, 4, 6, 10, 15 and 25 miles respectively. Where, however, a road leads away from the railway station, the radius of each circle, as it cuts the road, is doubled so as to allow for the value of the road in encouraging traffic on the railway.

The population-factor for each sub-area is then calculated separately, any portions of a circle or of its projection along a road which may come within a shorter distance of any other station either on the proposed railway or on any existing railway being omitted. The population-factor in each sub-area is multiplied by the following factors:—

1st ring,	0—1 mile	radius	$\times 3$
2nd ,,	1—2 miles	,,	$\times 2$
3rd ,,	2—4 ,,	,,	$\times 1\frac{1}{2}$
4th ,,	4—6 ,,	,,	$\times 1$
5th ,,	6—10 ,,	,,	$\times \frac{1}{2}$
6th ,,	10—15 ,,	,,	$\times \frac{1}{4}$
7th ,,	15—25 ,,	,,	$\times \frac{1}{8}$
Allowance over 25 miles			$\times \frac{1}{10}$

The total figure thus obtained represents the population-factor

of the station which is being considered, and, in order to obtain the probable coaching earnings, this factor is first multiplied by Rs. 0·7 (Rs. 1 = 1*s.* 6*d.*) which is an average figure of coaching earnings per unit of the population-factor obtained from the actual coaching earnings of a number of stations on the Great Indian Peninsular Railway, and the result is multiplied by one of the following coefficients depending on the nature of the station :—

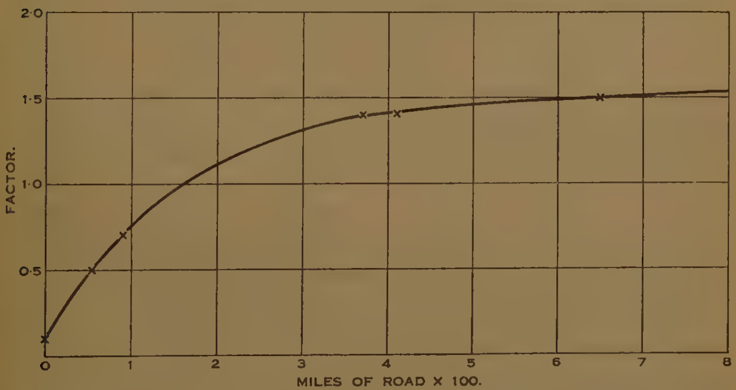
1·4 for stations at which mail and express trains stop.

1·2 for stations which serve a district or divisional headquarters or the capital of a native state.

0·7 for a military station or railway colony where the earnings are found to be lower, due, presumably, to the cheap tickets and free passes issued; wayside stations with no metalled cart roads leading up to them; and all stations on branch lines.

To the above figures must be added the earnings from any special coaching traffic expected which is not dependent on the

Fig. 1.



normal population of the area, such as pilgrim traffic. Where large towns are connected by roads with the proposed railway, the existing motor traffic should be estimated and the value of it deducted from the earnings of these stations, as given by the formula. It should be noted that, in the construction of a branch line, the area served by the existing railway will be reduced by the area served by the new railway; this will result in decreased earnings from some of the stations on the existing line, but the loss should be more than counterbalanced by the earnings from the new stations.

Goods traffic may be divided into traffic from special industries and general traffic. The revenue from special industries, such as coal and other mines, stone and lime quarries, cement factories, cotton mills, etc., must be estimated separately. The general goods earnings can be obtained from the following formula :—

General goods earnings $= P\sqrt{\frac{L}{100}}\,[0{\cdot}1 + R]$, where P denotes the population-factor of the station as given for coaching earnings, L denotes the average cost of land per acre in the area served by the station (roughly as given in the estimated cost of land for the proposed railway), and R denotes a coefficient depending on the number of miles of metalled road in the area served by the station. *Fig. 1* shows a curve utilized for obtaining this coefficient, and is based on figures worked out for a selected area on the G.I.P. Railway.

The Author claims for the formula not only that it gives a more reliable means of calculating the gross earnings of a project than those usually adopted in India, but also that it is, as far as he knows, the only formula that has been proposed by which the location engineer can calculate the effect on traffic earnings of altering the alignment and position of stations.

Paper No. 5125.

"Some Canadian Wharf-Structures of Steel Sheet-Piling."

By Robert Ferguson Legget, M. Eng.
Assoc. M. Inst. C.E.

(*Ordered by the Council to be published in abstract form.*) [1]

The Paper describes six wharf-reconstruction projects, carried out in widely differing parts of the east of Canada, which utilized interlocking steel sheet-piling as the main permanent unit of construction. It commences by commenting upon the attention now being devoted to that type of structure, known variously as the bulkhead or flexible type of retaining wall, and in the description of the six works, each of which has some special feature by reason of its location, reasons for the selection of this material and details of the cost are given.

As all the works described in the Paper were carried out for the Department of Public Works, Canada, a brief description of this interesting branch of the Dominion Government is presented. In more recent years many of the original duties of the Department have been delegated to other agencies (several going under provincial jurisdiction in 1867) and a summary of this development is given.

Reconstruction of Protection Jetties at L'Anse au Beaufils, Quebec.—At l'Anse au Beaufils, on the southern coast of the Gaspe peninsula, is a small enclosed fishing harbour. Its entrance is protected by two jetties, parts of which were completely demolished by a phenomenal storm in October, 1933. At that time, forty fishing boats were sheltering in the harbour, and reconstruction of the entrance had therefore to be completed before the break-up of the sea-ice in the following spring.

For this reason, and because of the economy of the requisite design, the use of interlocking steel sheet-piling was adopted for the reconstruction operations, which were successfully completed in the time available. A light section of piling (weighing 18·50 lbs. per square foot, with a section-modulus of 7·82 inch^3 units per foot of wall) was driven as two walls on either side of what remained of the

[1] The MS. and drawings can be seen in the Institution Library.—Acting Sec. Inst. C.E.

wrecked timber-cribwork jetties, only parts of which had to be removed. The walls were tied together by a system of tie-rods, connecting the continuous walers consisting of two channel sections. Copper-content steel was used for the work. Some comments are made on certain practical aspects of the exposure of steel-piling walls to the action of the sea.

Reconstruction of Wharf at Pointe au Pic, Quebec.—At Pointe au Pic, 86 miles below Quebec City on the north shore of the river St. Lawrence, is an important wharf serving both summer and winter traffic. Of substantial timber-cribwork construction, its substructure had deteriorated to such an extent that rebuilding became necessary, and in 1934 a contract was awarded by the Department for this work. Advantage was taken of the opportunity to enlarge the head-block of the wharf, and by the use of steel sheet-piling this was done with a minimum of disturbance of the old cribwork. The design adopted contemplated future dredging at the site of the wharf, and due to this feature, and the large tidal range, the use of 75-foot piles of a heavy section (weighing 48·74 lbs. per square foot, with a section-modulus of 55·1 inch^3 units per foot of wall) was necessitated. Driving was wholly in compacted sand, and, although hard, it caused no difficulty. The driven piles were secured to concrete anchor-walls by a tie-rod system, rods being attached to alternate piles with no continuous waler. Rock filling was used, and a tar-macadam finish provided for the wharf surface.

Reconstruction of Wharf at Fredericton, New Brunswick.—A wharf on the Saint John river, which consisted of an old timber-cribwork structure, was in such a condition that immediate reconstruction was imperative. A simple cross-sectional design utilizing steel sheet-piling fitted well with local conditions, and, was adopted, as it involved little disturbance of the old structure and proved to be most economical. A light section of piling was driven in a continuous wall immediately in front of the old structure, and tied back to concrete anchor-blocks by steel tie-rods connecting to a continuous channel waler. In elevation, the wharf has an interesting sloping profile to render it available for use at all stages of the river.

Reconstruction of Wharf at Port Hope, Ontario.—Wharves constructed of timber cribwork had in the course of time deteriorated so seriously that reconstruction was necessary, and in 1931 a start was made, utilizing steel sheet-piling (weighing 24·98 lbs. per square foot, with a section-modulus of 15·80 inch^3 units per foot of wall) driven as a simple wall in front of the old structure and tied back, through a channel-waler system and tie-rods, to a series of concrete anchor-blocks. The work contained one unusual feature in that for a part of the total length of wall there was insufficient unconsolidated

material overlying solid rock to give the necessary toe-hold. Cast-steel blocks were therefore secured to the bottom of every fourth pile, through holes in which a drill cored out holes in the rock into which steel dowels were inserted by divers and grouted into place.

Construction of Wharf at Oshawa, Ontario.—Construction of a new wharf-wall in the harbour of Oshawa presented special problems in that the site of the wharf extension was dry ground at an average elevation of several feet above lake water-level. By utilizing a design featuring a bulkhead wall of steel piling (weighing 31·74 lbs. per square foot, with a section-modulus of 25·35 inch^3 units per foot of wall) it was possible to drive the piling and to complete the superstructure and anchorage system, backfilling up to final grade before carrying out the necessary dredging in front of the new wall. The design adopted has a further interest in that it features a relieving platform which shields the wall of steel piling from the full active pressure of the fill it retains. The economies of the design adopted are discussed.

Reconstruction of Wharf at Keewatin, Ontario.—This was another old timber-cribwork wharf. The lake-bed at the site consists of a glaciated granite surface under a deposit of river silt and saw-mill debris. No toe-hold could therefore be obtained, but a bulkhead-wall design was evolved utilizing two channel-waler supports, one being under water. The latter was fitted to the piling, arranged in panels, before being lowered into the water, and in this way the entire main wall of the wharf was successfully rebuilt without the aid of divers. The design is described in detail, and the possibilities which it presents are discussed.

In an Appendix a short note is added regarding the main aspects of design work for bulkhead walls.

Paper No. 5128.

"Roads and Road Transport in Tanganyika Territory."

By William Harley McLuckie, M. Inst. C.E.

(Ordered by the Council to be published in abstract form.)[1]

General Description.—Tanganyika Territory lies on the east coast of Africa between 1 degree and 12 degrees south of the equator. It has an area of 365,000 square miles and a population exceeding 5,000,000. The principal activity is agriculture, and in recent years gold-mining has become important. The climate varies considerably and the Territory is not generally well watered. Its surface contains several large lakes and mountains. Other notable features are the long continuous scarps which cross the direction of transport to the Indian Ocean and restrict the possible alignments for railways. The scarps and mountains necessitate severe gradients for some of the roads.

The railway system consists of two unconnected metre-gauge single lines of a total length of 1,377 miles running westwards from the coast, one near the northern boundary, with a branch connecting with the Kenya and Uganda Railway, and the other centrally through the Territory to lake Tanganyika, with two branches running north, one of which reaches lake Victoria.

Apart from town and village roads and tracks the total length of the road system is 13,984 miles. Light motor transport over the greater part is considerable in the dry season, whilst over more than 4,000 miles comparatively heavy transport, with 2- and 3-ton lorries, is possible in the dry season and light motor transport practically throughout the year. 6,807 miles of village roads and tracks serve purely native purposes.

Construction and Maintenance of Roads.—The Paper describes generally the work of surveys, which are carried out principally to locate and demarcate the best routes between termini and to obtain information necessary to organize construction and order bridge materials from Britain. Existing maps are meagre in detail and considerable preliminary reconnaissance on foot is necessary. Ridge alignments are sought generally. The Territory is well covered with bush and the clearing which is necessary for construction is carried out to a width of 50 feet. Machinery has not been found economical in clearing or in removing ant-heaps. The cost of

[1] Type-litho copies of the full Paper can be obtained on loan from the Loan Library of The Institution; a limited number of type-litho copies are also available, for retention by members, on application to the Acting Secretary.

clearing has varied between £2 and £12 per mile, and the cost of reducing ant-heaps has reached £7 per mile.

Information is given concerning the formation and drainage of the roadways which are 16 feet in width and raised 6 inches above the general surface-level. Superelevation and widening are provided on curves with radii less than 600 feet. On steep side-sloping ground the formation is constructed with a cross-fall of 1 in 16 to the hillside. Road-grading machinery has not been used extensively and its use cannot replace more than one-quarter of the labour force normally required. The cost of formation and drainage varies upwards from £20 per mile, an average figure, inclusive of all earthworks, being about £55 per mile. Masonry and concrete culverts are most economical where suitable materials and foundations are available; but round corrugated-iron culverts are used most generally, their flexibility in soft and swampy ground being advantageous.

Many of the rivers flow only in the wet season and then rise and fall with such rapidity that the use of drifts is at present not unsatisfactory and is much more economical than bridging. Bridges are now constructed in accordance with standard types prepared by the Crown Agents for the Colonies, but in the past various types were used. Natural conditions permit considerable use of short spans and piers which are most economical, but the erection of larger spans has not been difficult.

Murrum gravel, decomposed rock and the material from ant-heaps are used for surfacing when such is essential, and 3-inch layers, repeated when necessary, are generally applied, though on occasion thinner layers have been found to provide the stability desired. Surfacing, 6 inches deep, has cost about £200 per mile, the greater part of the cost being due to transport of material.

In recent years average earth-road construction-costs have been as follows:—

Earthworks	£60 per mile
Structures	£90 ,,
Staff	£70 ,,
Plant	£20 ,,
Contingencies	£10 ,,
Total	£250 per mile

During the last 10 years approximately 1,500 miles of road, including 8,500 linear feet of bridges, have been constructed.

European supervision at a rate of 1 per 200 natives employed is generally required, though on occasion good foremen can supervise 500 men. The largest requirement of plant is light motor transport, an average of one lorry to 200 natives being required. Natives are paid from 6*d*. per day upwards and their wages amount to 35 per cent. of the whole cost. Materials and plant cost a similar pro-

portion. Roughly 100 natives are required to construct 1 mile of road per month. Maintenance is carried out by labour employed continuously over the principal roads at a strength of 1 man per mile and at a cost of £10 per mile per annum.

Expenditure on and Revenue from Roads.—Information is given of the expenditure incurred on roads which is provided from general revenue and from loans. In all 4 per cent. of the total expenditure of the Territory and 29 per cent. of the expenditure on public works has been expended on roads. Revenue derived from the road system is obtained from licences for vehicles and customs duties on vehicles, their fuel and spare parts, and it is stated that the portion of the general revenue derived from road use has provided for the greater part of the expenditure from revenue, and at the present time is exceeding the annual amount required.

Highway and Traffic Laws.—General powers for the construction and use of roads are provided by a Highway Ordinance, which also provides for the use of the roads to be restricted and prohibited during the wet season. A Traffic Ordinance governs the condition and use of vehicles on the roads and limits their maximum weight and axle-weight on main roads to 10 tons and 2½ tons respectively, and on district roads to 3 tons and 1½ ton respectively. A Control Ordinance prohibits the use of vehicles carrying goods on roads competitive with the railways.

Road Transport.—Within the last 10 years light motor transport has replaced the porters and ox transport formerly used, the number of lorries and cars having increased five and twelve times respectively. The intensity of traffic is increasing by 33 per cent. annually and at present averages 15 vehicles per mile per day over the principal roads. The road cost per ton-mile is probably less than ½*d.*, whilst it is stated that the costs of twenty-two selected Government vehicles during 1935 averaged 4·35*d.* per ton of carrying capacity per mile, and in advantageous circumstances could be reduced to the vicinity of 3*d.*

Reference is also made to future road development and to road and rail competition, and it is stated that as both the railways and the roads have been provided at the expense of the State, the co-ordination of traffic on them is in the public interest. The whole question of competition has been the subject of an expert investigation recently and a Report on the co-ordination of transport in Kenya, Uganda and the Tanganyika Territory, by Brigadier-General Sir Osborne Mance, has been issued recently. At present the position is not as acute as it was during the years of depression.

ADDITIONAL ORIGINAL COMMUNICATIONS

RECEIVED BETWEEN THE 1st APRIL, 1935, AND THE 31st AUGUST, 1937.[1]

AUTHORS.

Addison, Herbert. No. 5,081.—Supplementary Notes on Flow Through Model Sluices.

Ahmed, S. B. No. 5,147.—Theory of Earth-Pressure as Applied to Retaining-Walls.

Barnes, A. A. No. 5,067.—Fluid Flow: Part I. The New Dimensionless Ordinate. Part II. The Law of the Eddies.

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Binnie, A. M. No. 5,141.—The Standing Wave in a Uniform Horizontal Channel of Rectangular Cross-Section.

Blench, Thomas. No. 5,115.—Turbulent Flow in Channels.

Blench, Thomas. No. 5,140.—Energy Theory of Turbulent Flow of Liquids.

Blofield, G. C. No. 5,083.—Reconstruction of the Mocoreta and Timboy Bridges, Argentine North Eastern Railway.

Buckton, E. J., and Fereday, H. J. No. 5,150.—The Reconstruction of Chelsea Bridge.

Burls, G. A. No. 5,133.—A General Graphical Solution of the Problem of the Rough Inclined Plane.

Burns, R. V., and White, C. M. No. 5,127.—The Protection of Dams, Weirs and Sluices against Scour.

Cherre, V. No. 5,134.—Revolving Sluice-Gate (System V. Cherre).

Colebrook, C. F., and White, C. M. No. 5,137.—Reduction of Carrying Capacity of Pipes with Age.

Cornish, R. J. No. 5,092.—The Discharge of Small Submerged Sharp-Edged Orifices.

[1] Available for reference in the Library; includes Papers awaiting publication.

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Currie, R. M. No. 5,095.—A Waterworks Pumping-Plant in China.

Davies, D. Tudor. No. 5,119.—The Construction of Port Sudan Harbour.

Davies, H. V. No. 5,131.—A Simple and Efficient Type of Sewage-Disposal Works for Small Communities, with Special Reference to the Performance of the "Pruss" Biological Filter when dealing with a Septic Tank Effluent at Bloemfontein, South Africa.

Donkin, Thomas. No. 5,079.—The Effect of the Form of Cross-Section on the Capacity and Cost of Trunk Sewers.

Dunkley, F. G., and Fancutt, F. No. 5,112.—The Engineer and the Paint Technologist.

Ellis, S. J. No. 5,143.—The Total-Heat-Entropy Diagram for Diphenyl.

Ellson, George. No. 5,146.—Dover Train-Ferry Dock.

Elsden, Oscar. No. 5,100.—Investigation of the Outer Approach Channels to the Port of Rangoon by Means of a Tidal Model.

Fancutt, F. No. 5,111.—Painting as it Affects a Railway Engineer.

Ferguson, Clyde. No. 5,135.—A Note on Foundations.

Follenfant, H. G. No. 5,052.—A Pre-cast Reinforced-Concrete Underline Railway Bridge.

Fortune, L. E. No. 5,030.—The Reconstruction of a Small Sewage-Disposal Works.

Gelson, W. E. No. 5,126.—Some Experiments on Locomotive Springs, with Reference to Bridge Impact Allowances.

Gibson, A. H. No. 5,110.—An Experimental Investigation of the Effect of Bridge Piers and Other Obstructions on the Tidal Levels in an Estuary.

Gumersall, G. J. No. 5,053.—Mathematics of the Multiple Portal, as Applied to Wind Calculations for large Power-Stations and Mill Buildings.

Higgins, A. L. No. 5,129.—The Adjustment of Traverse Surveys.

Hiley, Alfred. No. 5,093.—Some Considerations on the Bearing Capacity of Foundations and Sub-Soil Values.

AUTHORS.

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ENGINEERING RESEARCH.

SIR ALEXANDER GIBB'S ADDRESS TO THE BRITISH ASSOCIATION.

The Institution notes with pleasure that its President, Sir Alexander Gibb, has been chosen as this year's President of Section G (Engineering), of the British Association. The subject of his Presidential address was "Research in Engineering."

Engineering, in its early days regarded as an art, passed through a period of empirical practice to its final stage of applied science. That it was a science was still not sufficiently recognized, nor that without research were progress and improvement possible. The encouragement of research and the advancement of useful knowledge were among the objects of the foundation of The Institution in 1818.

The President briefly traced the history of research showing how very recent was its development. The Imperial College of Science, South Kensington, owed its origin to a financial surplus from the Great Exhibition of 1851. The Royal Society, subjected in its early days to much derision, received from the Government in 1850 an annual grant of £1,000, which was increased in 1877 to £5,000 a year, for research in science generally, but original research continued to be mainly left to the individual at his own expense. In 1899, as the result largely of an agitation originating at meetings of the British Association, the National Physical Laboratory was founded. In 1902 the British Engineering Standards Association was established by the co-operation of the Institution of Civil Engineers, the Institution of Mechanical Engineers, the Institution of Naval Architects, and the Iron and Steel Institute. The need for a wider organization linking more definitely science and engineering research and industry led to the establishment, in 1915, of the Department of Scientific and Industrial Research under the control of a Committee of the Privy Council, whilst in 1918 the National Physical Laboratory became part of the newly-created Department, which was now the common focus all the research carried out in the country. That was largely accomplished by the encouragement of the formation of Research Associations, self-governing bodies formed on a national basis in various industries and working in co-operation with the Department, which gave financial support on a scale dependent upon the income otherwise provided. Of equal importance was the work carried on under the ægis of the great scientific Institutions. Valuable research work was carried out by the research departments of the Universities,

and finally the more important and wealthier business firms had their own private research laboratories. Whilst those were independent units, they could work in contact and correspondence with the Department of Scientific and Industrial Research, if they so desired.

The President then traced the growth of the parallel research organizations in the United States, where the National Academy of Science, founded in 1836, was the counterpart of the Royal Society; the National Research Council, formed in 1916, compared with the Department of Scientific and Industrial Research, and the National Bureau of Standards, formed in the first few years of the century, corresponded to the British Standards Institution. A particular form of American research institution was exemplified by the Mellon Institute. That had a limited membership in which one representative only of each class of interest was admitted, and resulted in a broadening of the basis of research. In Canada the National Research Council was carrying out a wide programme of practical research.

The financing of research originally devolved upon the individual, and later much assistance had been and was still being given by public-spirited benefactors. The cost should, however, be borne by those who benefited, namely the community and industry, and that the latter was now appreciative of the value of research was exemplified by the increase of 40 per cent. in 3 years in its support of research associations, which in 1935–6 totalled £232,468. Even so the position was not yet satisfactory, full use not being made of the facilities offered by the Government.

There was no general national plan for research as obtained in totalitarian States. It was the more necessary, therefore, that democratic countries, too, should organize and co-operate more closely than before. Groups of unrelated, often competitive, bodies could not be really effective, and in the President's opinion it was inevitable that sooner or later every research organization would have to be linked in some way to a central controlling body. Only thus could hopeless overlapping and waste of energy, time and money be avoided.

No less necessary was some form of centralization in respect of publications. The vast numbers of publications dealing with research and the innumerable channels through which the results of a research were broadcast made it a matter of extreme difficulty to obtain the latest information on any particular subject. There should be a clearing-house of engineering information which would collect, collate and make immediately available all new data discovered. If such an organization on a world-wide basis were impossible, the establishment of at least a clearing-house system in co-operation with the Department of Scientific and Industrial Re-

search could not be impossible, and the expense involved would be relatively small. The Department with its research associations was the ideal nucleus for such a service, but engineering had to work out its own scheme.

At the moment the two outstanding questions in the engineering world were the co-ordination of effort and the promotion of intensive research.

THE INSTITUTION RESEARCH COMMITTEE.

Sub-Committee on Reinforced-Concrete Structures for the Storage of Liquids.

This Sub-Committee, the formation of which, under the Chairmanship of Mr. Halcrow, was announced in the Journal for November, 1935,[1] was appointed to prepare recommendations for a Code of Practice for the design and construction of reinforced-concrete structures for the storage of liquids. At an early date the Sub-Committee was sub-divided into four Panels dealing with :—

(1) General considerations,
(2) Materials,
(3) Design,
(4) Construction.

Meetings of the Panels have been held at regular intervals and good progress has been made.

The Design Panel, however, found that information was lacking concerning the behaviour of bituminous joints such as may form the horizontal sliding joint at the base of a reservoir wall or a vertical contraction or expansion joint between sections of the wall. Whilst it was felt that, in the present state of knowledge, no specification for bituminous materials could be included in the recommendations for a Code, it was agreed to be desirable that tests should be carried out to throw some light on the mechanical behaviour of such material.

Arrangements have accordingly been made for the carrying out of tests designed to afford some guidance on this matter, and Professor R. G. H. Clements has kindly undertaken to carry out such a research of a few months' duration at the City and Guilds College. It is proposed, in particular, to study the sliding of a bituminous joint subjected to normal pressure so as to determine the equivalent friction of the joint and to obtain information concerning the behaviour of bituminous jointing materials in a joint subject to alternate expansion and contraction.

[1] Journal Inst. C.E., vol. 1 (1935–36), p. 44 (November, 1935).

It should be mentioned that there is ample scope for a comprehensive research into the behaviour of bituminous and other jointing materials for water-retaining structures. Such a research would, however, be costly and of considerable duration. It was for this reason, and in order to permit the Sub-Committee to conclude its work, that the above-mentioned tests alone were decided upon.

REPORT OF THE BUILDING RESEARCH BOARD FOR THE YEAR 1936.

Developments of outstanding interest mentioned in the Report, which has recently been published, are the coming into operation of the Fire-Testing Station at Elstree and the new Heating Laboratory at the Building Research Station. A note on the Fire-Testing Station appears on p. 584.

An interesting retrospect of the Station's activities since its construction in 1935 has been included. The Director traces the evolution of the organization to its present sub-division into three main sections: research, inquiries and special investigations, and library and publications. Under the last-mentioned heading, the monthly publication of Building Science Abstracts has been developed. Of particular interest to engineers are the researches into the properties of concrete. An early investigation into the permeability of concrete indicated the need for a series of fundamental investigations in which attention could be concentrated on one variable at a time. The first step was the construction of a laboratory in which temperature and humidity conditions could be controlled within close limits. Then an investigation into methods of testing Portland cement was begun and a standard 1 : 3 mortar-cube test was developed. By subsequent improved methods uniformity of results is ensured by vibration. A search for an ideal aggregate-grading showed that within practical limits the strength was independent of grading provided that the same water/cement ratio is used and that the concrete is thoroughly consolidated. This led to the study of workability and culminated in the standardization of a compacting-factor test.

Research on reinforced concrete led to the drafting of the Code of Practice for the Use of Reinforced Concrete in Buildings. Many problems raised during the course of this work, such as redistribution of bending moments in reinforced-concrete frames, bending in reinforced-concrete columns, and cracking in reinforced-concrete structures, have since been investigated.

Other sections of particular interest deal with the work of the

Steel Structures Research Committee and the research on pile-driving. The work on soil-mechanics is described from its commencement with the work of Professor C. F. Jenkin on dry sand, culminating in his revised wedge theory, to the present work on clays and other soils which is being carried out in conjunction with The Institution.

The work during the year includes a method of cleaning building stones by means of a water-jet, thus avoiding disadvantages inseparable from the use of chemicals. Work is continuing on the weathering and other properties of asphalts and bitumens. Research on cements includes a study of the carbonation of unhydrated Portland cement, special cements for large dams (jointly with The Institution), pozzuolanas, curing of cast-concrete products, asbestos cement, and so on. Stability of foamed slag as an aggregate has been studied. Other materials studied are limes and plasters, clay building-materials and paints, whilst a study has been made of the effect of frost on building materials.

Research on structures and on strength of materials includes a study of eccentrically-loaded reinforced-concrete columns, and the prevention of cracking by pre-tensioning of reinforcement. It is found that less than two-thirds of the nominal pre-tension is effective owing to creep of the concrete. Research is being conducted into concrete road-slab design, and has emphasized the weakness of the corners of slabs. The relation between grading of aggregates and workability of concrete has been studied, using crushed granite, sand and gravel aggregates.

The work carried out in conjunction with The Institution on earth-pressures, pile-driving, vibrated concrete, and the action of sea water on reinforced concrete, has been described in previous Journals.[1] The research on voussoir arches carried out for the Station at the City and Guilds College by Professor Pippard has been described in the Journal.[2]

Tests on beams of both mild steel and high-tensile structural steel have been directed to the study of failure by crushing at the bearing, by vertical web-buckling and by shear failure. An investigation of existing road bridges has been made for the Ministry of Transport. The investigation on wind-pressures on structures has been completed, and, with the observations taken on the Severn bridge, information is available relating to wind fronts up to ½ mile.

A series of researches has been carried out concerning the efficiency of buildings from the standpoint of the user, and a description is given

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[1] See Indexes to the Journal, appearing in Vols. 3 (1935–36) and 6 (1936–37). (October, 1936 and October, 1937).

[2] Journal Inst. C.E., vol. 4 (1936–37), p. 281 (December, 1936), and vol. 6 (1936–37), p. 5 (June, 1937).

of the new Heating Laboratory. Tests are in progress on heat-transmission through various types of brick walls, heat-transmission through roofs, thermal transmittance of a window, the effect of moisture on the thermal resistance of insulating wall-boards, and the exclusion of solar heat. Research into ventilation has shown the value of banks of vanes or splitters in reducing the resistance at elbows in ventilating ducts. Work on acoustics has shown the importance of isolating a sound at its source. Tests have been carried out on "floating" slab floors and also on false ceilings.

Among the special investigations carried out may be mentioned the production of aggregate from clay and mud, steel-sheet structural floors and wood-cement partition slabs.

THE FIRE-TESTING STATION, ELSTREE.

The Fire-Testing Station erected by the Fire Offices Committee at Elstree was completed in 1935. By arrangement with this committee, the Fire-Testing Station is available to the Building Research Station for research on fire-resistance, for investigations for outside bodies, and for tests on proprietary structures where the Department's certificate of their fire-resistance is desired.

The essential equipment consists of three testing furnaces, together with a control room from which the temperatures are controlled and automatically recorded. In addition there is space for the construction and dismantling of test specimens, and the whole floor space is commanded by a 30-ton overhead travelling crane. The cylindrical furnace erected vertically is in two halves each mounted on a carriage which can be run back so as to allow a hose to be played on the heated specimen. This furnace is designed for the testing of columns up to 10-foot-long heated area. The column is mounted in the centre of the furnace in a special 500-ton testing machine. A panel furnace is provided for wall slabs. Wall slabs up to 10 feet square with edges either free or restrained are held vertically in a 500-ton testing machine. The furnace consists of a panel of burners likewise 10 feet square which is mounted on a carriage so as to enable it to be withdrawn from the test panel and a water-hose played on the latter. The floor furnace is capable of testing either floors up to 10 feet square or beams 10 feet long. In this case it is the test specimen which is removed before applying the hose test.

Under the British Standards Definitions (B.S.I. No. 476) specimens are classified according to their fire-resistance capacity into categories A, B, C, D, and E. The general test is to heat up the face of the specimen according to a standard heating-curve, which reaches a

final temperature of 1,200° C. after 6 hours. The specimen which is subjected to $1\frac{1}{2}$ times its design load, is heated for periods of 6, 4, 2, 1 and $\frac{1}{2}$ hours for categories A, B, C, D, and E respectively. In the case of slabs and floors, the temperature on the cold side must not exceed 350° F. On completion of the test period a fire-hose is applied uniformly over the heated surface until it is cooled.

Work has been carried out on floors consisting of timber, fillerjoist, hollow clay blocks with reinforced-concrete ribs, and reinforced-concrete slabs. Panels of light-weight concrete blocks, slabs of wood fibre and cement, and brick panels have also been tested.

RESEARCH IN ENGINEERING AT KING'S COLLEGE, NEWCASTLE-UPON-TYNE, AUGUST, 1937.[1]

The researches noted below are being carried out in the Departments of Mechanical, Marine and Civil Engineering, of Electrical Engineering and of Mining.

Civil Engineering.

Much work has been done on photo-elastic methods of stress-analysis. In addition to the determination of the stresses in simple frames the method has been applied to such practical problems as the analysis of the stresses in a cylinder-head casting and the stress-distribution in cases, such as angles subjected to bending, which are not amenable to mathematical treatment.

The internal stresses and strains in welded joints are being investigated. For this purpose an extensometer is not suitable as it cannot remain in situ during welding, and an optical comparator reading to $\frac{1}{200,000}$ inch has accordingly been developed. By exploring the longitudinal and lateral strains resulting from welding, and from subsequent relief of strain after cutting, the stress-distribution can be obtained. The effect of annealing on the stresses and strains is being studied.

Mechanical Engineering.

The equivalent length of a crankshaft for the calculation of torsional vibration is being studied experimentally.

A long series of researches has been made into the behaviour of piston rings, the relation between friction and cylinder-pressure and its variation with the design of ring, the amount and effect of air

[1] Under the Statutes made in accordance with the University of Durham Act, 1935, the College of Medicine and Armstrong College have been merged in King's College, Newcastle-upon-Tyne, in the University of Durham.

leakage, etc., having been investigated. Work is now held up pending the development of a suitable high-speed indicator.

The scavenging of the two-stroke oil engine is being studied, using water as the working fluid, the scavenging-efficiency being measured by the salt-solution method.

A research has been carried out with the object of investigating the effect of viscosity and pipe-dimensions on the velocity of pressure-waves in oil under pressures up to 6,000 lbs. per square inch in pipes of small bore. This is of practical significance in connexion with the oil-injection system of the diesel engine.

Marine Engineering.

The virtual increase of mass of a propeller, immersed in water, when the shaft is subjected to torsional oscillations has been ascertained in the case of a number of scale models. The interpretation of the results in terms of the prototypes is, however, difficult and inconclusive.

Electrical Engineering.

An electric miner's lamp has been developed which gives an indication of the presence of inflammable gas and, when a predetermined concentration has been reached, shuts off the light momentarily.

The variation of electrical resistance with thickness has been utilized to develop an instrument whereby the varying thickness of metal tubes and plates may be explored. By this means boiler and retort tubes may be surveyed in situ and the corrosion of ships' plates and tanks measured.

The precision measurement of very high voltages presents difficulties, and a method is being developed based upon the oscillation of a suspended ellipsoid in a uniform electrostatic field.

A fundamental research is being conducted into the relation between breakdown resistance of various gases and their molecular structure.

Mining.

Special attention is being given to the development of methods of geophysical surveying. The variation in electrical resistance of different strata is being utilized by the probe, the expanding traverse and the step methods to indicate their nature and location. Small-scale models have been used to facilitate the interpretation of field results. In connexion with magnetic surveying the use of the vertical-force magnetic variometer for the location of magnetic ores and the

correlation of underground and surface surveys by the magneto-theodolite have been studied. Work is being done in connexion with the Fuel Research Coal Survey of the Northumberland and Durham coal field.

The use of fans of air-screw form for obtaining increased efficiencies in mine ventilation has been studied.

Methods of cleaning and recovery of small coal are being investigated. By way of contrast another research is directed to the elimination of carbonaceous particles from shale so as to render it suitable for the manufacture of bricks.

The foregoing researches are being carried out under the direction of Professor C. J. Hawkes, M.Sc., Professor of Mechanical, Marine and Civil Engineering, Professor W. M. Thornton, O.B.E., D.Sc., D.Eng., Professor of Electrical Engineering, and Professor Granville Poole, B.Sc., F.G.S., Professor of Mining.

NOTES ON RESEARCH PUBLICATIONS.[1]

Engineering Materials: Properties and Testing.

Timber.

Department of Scientific and Industrial Research, Forest Products Research Record No. 18 deals with the causes of stain and decay in imported timber, and *No. 21* with the growth and structure of wood.

Bricks, Cement, and Concrete.

Work done at the Building Research Station on the effects of soluble salts in clay products is described in *Trans. Ceram. Soc.*, **36,** *233.*

The effect of the heat of hydration of a cement on the quality of a concrete is discussed in *Zement*, **26,** *87.* In *J. Am. Concr. Inst.*, **8,** *577*, the properties of cements and concretes containing fly ash are discussed. The following researches on aggregate and grading have been noted: The law of size-distribution and statistical description of particulate materials, *J. Franklin Inst.* **223,** *609*; The effect of the grading of the aggregate on the strength and workability of concrete, *J. Inst. Mun. & County Engrs.*, **64,** *114*; The workability of concrete and mortar, *Eng. News-Record*, **119,** *17*; Aggregate grading in relation to concrete mix design, dealing with aggregates

[1] The figure in heavy type is the number of the Volume; that in brackets the number of the Part; and that in italic type the number of the Page; in references to "Engineering Abstracts" the number of the Abstract is given.

up to 3 in. diameter, *Trans. Inst. Civ. Engrs. of Ireland*, **62,** *197.* In *J. Am. Concr. Inst.*, **8,** *339*, is a study of sub-aqueous concrete. In *Proc. 16th Annual Meeting* (*1936*) *U.S. Highway Research Board, p. 193*, is a Paper on the bond of vibrated concrete. The results of a research on transverse elasticity and creep of concrete are given in *Engineering*, **143,** *161.* The shrinkage of concrete and the use of expanding cements are dealt with in *Annales des Ponts et Chaussées*, **107**-*i*, *15* (*Eng. Abs.* **75,** 28); and in *J. Am. Concr. Inst.*, **8,** *327*, the effect of size on drying shrinkage is discussed. Permeability is dealt with in *Zement*, **26,** *93*, and on *p. 189*, an improved method for its determination is described. *Bulletin No. 53, Rensselaer Polytechnic Institute*, deals with the weathering resistance of concrete; and some tests on the effect of freezing on the permeability, strength and elasticity of concrete and mortars are described in *Am. Soc. Testing Materials, 1937 Preprint No. 55.*

Metals.

The mechanical properties of carbon steels, copper, and aluminium alloys broken in tension at very high speeds are described in *J. Inst. Metals*, **61,** *263.* A study of the effect of span on transverse test results for cast iron is given in *Am. Soc. Testing Materials, 1937 Preprint No. 31.* A method for detecting and locating laminations and flaws in steel plates by a study of the sand patterns formed at resonance is described in *J. Am. Acoustical Soc.*, **8,** *220.* The mechanism of film-formation in corrosion processes is discussed in *Chem. & Industry*, **56,** *751.*

The effect of surface condition upon the fatigue strength of aluminium wires is dealt with in *Zeitschrift Metall.*, **29,** *214.* *Research No. 18* of the *British Non-Ferrous Metals Research Assoc.*, on the properties of lead and lead alloys, gives a summary of the fatigue resistance of lead and lead alloys, and in *Research No. 58* published information on the creep of non-ferrous metals and alloys is reviewed.

Other Materials.

The physical properties of typical American rocks are discussed in *Bulletin 131, Iowa Engg. Expt. Stn.* A radial flow method for the measurement of permeability of porous media is given in *Phys. Rev.*, **51,** *684.* The colloidal nature of asphalt as shown by its flow properties is discussed in *J. Phys. Chem.*, **40,** *1133.*

Engineering Materials: Production, Manufacture, and Preservation.

Research on timber includes *Department of Scientific and Industrial Research Forest Products Research Record No. 19*, Methods of kiln

operation, and in *Trans. Soc. Chem. Ind.*, **56,** *202,* a new aspect of the action of timber fireproofing compounds.

The deposition of concrete treated by the vacuum process is described in *Stroitelnie Promishlennost,* **15,** (*3*), *44* (*Eng. Abs.* **75,** 29). The reconditioning of Portland cement deteriorated by prolonged storage by the addition of hydrochloric acid to the mixing water is described in *Cement,* **10,** *15.* The manufacture of cast iron to resist high temperatures is dealt with in *Revue Univ. des Mines et de la Métallurgie,* **80,** *281.*

Structures.

Mass Structures.

Geophysical exploration of foundation strata is explained in *Bauing.,* **18,** *271.* The development of soil mechanics is described in *The Structural Engineer,* **15** (*New Series*), *327.* A preliminary investigation into the subject of foundations in the black cotton and *Kyatti* soils of the Mandalay District, Burma, is summarised in *Proc. of the International Congress on Soil Mechanics and Foundation Engineering* 1936, **3,** *242.* It is suggested, *inter alia,* that cracking of buildings is reduced by increasing the intensity of pressure on the footings; thus in the cases considered reduction of area so as to give more than 1 ton per square foot is found to be desirable. The vertical pressures beneath a spread foundation are obtained graphically in *Proc. Am. Soc. Civ. Eng.,* **63,** *669.* Soil consolidation by means of a vibrator is dealt with in *Bautech.,* **15,** *219* (*Eng. Abs.* **75,** 45). A mathematical investigation of the rational design of pile foundations is given in *U.S. Department of the Interior, Bureau of Reclamation, Tech. Memo. 531. Punjab Irrigation Research Inst. Research Pubn.,* **2,** *No. 16,* gives the general theory of the gradient of pressure under a structure on permeable foundations, with applications to the evaluation of the gradient at exit for some standard cases, and *No. 19* discusses the relative efficiency of a vertical sheet-pile under a flush floor. A mathematical solution for the predetermination of the temperature-rise in large masses of poured concrete is given in *Zement,* **26,** *158.*

Framed Structures.

The solution for the effect of two isolated forces on the elastic stability of a flat rectangular plate is given in *Proc. Cambridge Phil. Soc.,* **33,** *325.* In a study of relaxation methods applied to engineering problems, the deflexion of beams under transverse loading is dealt with in *Proc. Roy. Soc. Series A,* **161,** *155,* and on *p. 197* a solution is given for the elastic stability of a thin twisted strip. Formulas

are derived for the bending strength of thin-walled cylindrical tubes in *Report No. 9, Structural Research Lab., Roy. Tech. Coll., Copenhagen ;* failure is found to be due to instability and the formation of lobes. A theory is evolved for beams resting on a yielding foundation in *Proc. Nat. Acad. Sciences, U.S.A.*, **23,** *328.* The effect of shear on the frequency of transverse vibration of a loaded fixed-free bar is discussed in *Phil. Mag.*, **23,** *1129.* The method of basic co-ordinates whereby it is possible to design plane frames without the use of statical formulas is explained in *Mitteilungen aus dem Institut für Baustatik an der Eidg. Technischen Hochschule, Zurich, No. 7.* Recent developments in photo-elasticity are described in *Annali dei Lavori Pubblici*, **75,** *90* (*Eng. Abs.* **75,** 3), and the photo-elastic analysis of T-tail stresses as in the pole-pieces of electric generators is discussed in *J. Franklin Inst.*, **273,** *715.*

Research in reinforced concrete includes experiments concerning the strain of the reinforcement of reinforced-concrete beams due to the shrinkage of the concrete, *Beton und Eis.*, **36,** *175* (*Eng. Abs.* **75,** 52) ; An experimental study of bond stress, *Proc. 16th Annual Meeting* (*1936*) *U.S. Highway Research Board, p. 96* ; The design of reinforced-concrete members under flexure or combined flexure and direct compression, *J. Am. Concr. Inst.*, **8,** *483* ; and in the same journal, *p. 459,* Rapid and long-time tests on reinforced-concrete knee-frames. A study of expansion-joint fillers, with particular reference to concrete roof-slabs, is given in *U.S. Nat. Bur. Stands., Tech. News Bull.* (*240*) *37.* A discussion of general principles in connection with fire resistance of buildings is given in *The Structural Engineer*, **15,** *257.*

Transformation, Transmission, and Distribution of Energy.

Department of Scientific and Industrial Research, Fuel Research Tech. Paper No. 45, has been published, dealing with the hydrogenation-cracking of tars, Part III : the effect of certain variables in a continuous plant. The injection of petrol in an internal-combustion engine is studied in *Comptes Rendus*, **204,** *1316* (*Eng. Abs.* **75,** 78) ; and the ignitability of diesel-engine fuels in *Auto. Zeit.*, **40,** *195* (*Eng. Abs.* **75,** 80). Boiler water problems are dealt with in *J. Inst. Heat. and Vent. Engrs.*, **5,** *149.* In *J. Inst. Elec. Engrs.*, **80,** *567,* is given an experimental investigation of the theory of eddy-currents in laminated cores of rectangular section, and on *p. 647* nickel alloys of high permeability, with special reference to Mumetal, are discussed ; in the same journal, *Vol.* **81,** *p. 145,* the lay-out and rupturing capacity of protective devices in motor circuits is considered ; on *p. 256* is a Paper on the metering of mercury-arc rectifier

supplies and outputs; and on *p. 277* a variable air condenser with adjustable compensation for temperature is described. The following researches in *Archiv Elek.*, **31,** have been noted: *p. 141* (*Eng. Abs.* **75,** 23), The rupture-gradient of liquid and solid benzol; *p. 166* (*Eng. Abs.* **75,** 101), Rupture voltage reduction at high frequency; *p. 179* (*Eng. Abs.*, **75,** *98*), The rupture voltage of flowing oil and the increase of the rupture field-strength by electric filtration; *p. 186* (*Eng. Abs.* **75,** *103*), The distortion of surge-waves in short transmission-lines; *p. 197* (*Eng. Abs.* **75,** 102), The rupture of stratified oil-air dielectrics with impulse and alternating voltages; *p. 282* (*Eng. Abs.* **75,** 100), Electric rupture of various gases under high pressure; *p. 287,* Effect of earthing- and compensating-coils upon the mutual influence of earthed three-phase systems; *p. 338,* The problem of corona-damping of surge-waves. The dielectric strength of non-inflammable synthetic insulating oils is dealt with in *Elec. Eng.*, **56,** *671.* The ageing of copper conductors traversed by electric currents is discussed in *Comptes Rendus,* **204,** *1715.*

MECHANICAL PROCESSES, APPLIANCES, AND APPARATUS.

The following researches in welding have been noted: Stress-distribution in fillet welds—a review of the literature to date, *Welding J.*, **16** (*5*) (*Suppt. 1*); The welding of copper and its alloys, *J. Am. Weld. Soc.*, **16** (*2*) (*Welding Research Suppt. 7*) and (*3*) (*Suppt. 33*) (*Eng. Abs.* **75,** 113); The welding of cast iron, *J. Am. Weld. Soc.*, **16** (*3*) (*Suppt. 2*) (*Eng. Abs.* **75,** 111); The welding of thick aluminium sheets, *Ver. deu. Ing.*, **81,** *433* (*Eng. Abs.* **75,** 115); Welding stresses in vessels subjected to internal pressures, *Stahl und Eis.*, **57,** *389* (*Eng. Abs.* **75,** 114); Electric welding of steel bridges, giving the experience of the Department of Main Roads, N.S.W., *J. Inst. Engrs. Australia,* **9,** *173.*

In the *Report of the Food Investigation Board for 1936* research into thermal conductivity, heat-transfer, refrigeration, evaporation, and control of humidity is described.

The lubrication of journal bearings in oxidizing conditions is discussed in *J. Inst. Petroleum Technologists,* **23,** *350.*

SPECIALIZED ENGINEERING PRACTICE.

Transport.

In *Publication No. 4* of the Institute of the German Research Association for Soil Mechanics at the *Technische Hochschule,* Berlin, the use of dynamic foundation soil investigation and the behaviour of sand with changes in loading and movements of ground water are discussed. Soil-consolidation by vibrators is dealt with in *Bautech.*,

15, *219.* The improvement of earth and worn-out gravel roads by the use of cut-backs and road oils to stabilize the soil is discussed in *Proc. 16th Annual Meeting (1936), U.S. Highway Research Board, p. 359*; and on *p. 69*, safe friction factors and superelevation design are dealt with in the Report of the Committee on relation of curvature to speed. In *Bull. No. 110, Permanent International Assoc. Road Congresses, 26th year, p. 92*, the lay-out and construction of roads according to the new standards recommended is described, and on *p. 100* is an article on road design and road safety. A study of the friction between road surface and motor vehicles is given in *Betonstrasse*, **12** (*6*), *125* and (*7*) *139.* Research on concrete roads is dealt with in *Betonstrasse*, **11,** *99.* In *No. 110, Permanent International Assoc. Road Congresses, 26th year, p. 79*, is a description of American research concerning joints, and in *Eng. News-Record*, **119,** *72*, a new expansion joint for concrete pavement is discussed. The mechanical properties of bituminous surfacing materials under constant stress are considered in *J. Soc. Chem. Ind.*, **56,** *146T.*

Research on railways includes: A study of the development of cracks in the wheel-seats of axles within the hubs of wheels due to the effect of radial compressive stress between shrunk or forced-on wheel and axle, *J. Inst. Engrs. Australia*, **9,** *215*; the spalling of the tire-surfaces of railway-wheels, *Stahl und Eis.*, **57,** *533* (*Eng. Abs.* **75,** 19); High-speed passenger trains, *J. Western Soc. Engrs.*, **42,** *47*; Stresses in railway-track due to high speeds, *Rev. Gén. Chemins de Fer*, **56,** *345* (*Eng. Abs.* **75,** 131); and Lubricants and "false-brinelling" of ball and roller bearings, *Mech. Eng.*, **59,** *415* (*Eng. Abs.* **75,** 124).

The following researches on marine transport have been noted: The influence of surface roughness on ship resistance, *Schiffbau*, **38,** *135* (*Eng. Abs.* **75,** 141); The influence of form upon the frictional resistance of models towed under water, *Schiffbautech. Gesellschaft*, **38,** *177*; and in the same journal, *p. 260*, Ship-stabilization; The resistance of a ship among waves, *Proc. Roy. Soc. Series A*, **161,** *299*; Open test results of four-bladed propellers, *Het Schip.*, **19,** *84* (*Eng. Abs.* **75,** 90); Experimental results for a series of three-bladed model propellers in open water, describing work carried out at the Wm. Froude Laboratory, *Trans. Liverpool Engineering Soc.*, **58,** *214.*

The following Aeronautical Research Committee Reports and Memoranda have been noted: *No. 1731*, Plane-table method of measuring take-off and landing flight-paths; *No. 1734*, Full-scale tests of Hartshorn ailerons on a Bulldog; *No. 1735*, Wind-tunnel tests on slotted flaps on a low-wing monoplane: flap angle 0 deg. to 90 deg.; *No. 1736*, Wind-tunnel tests to determine the efficiency of an airscrew working in front of a thick-section wing; *No. 1751*,

A study of the flexural axis positions for certain box sections; *No. 1755*, Notes on stubs for seaplanes; *No. 1756*, The distribution of stress in monocoque wings; *No. 1757*, A general method of calculating the effect of axial constraint on torsion on different forms of two-spar, skin-covered wings; *No. 1759*, Some applications of conformal transformation to airscrew theory; *No. 1762*, The R.A.E. mark Va torsiograph; *No. 1763*, A note on roughness; *No. 1768*, Some experiments with cascades of aerofoils; *No. 1771*, Tests of four airscrew sections in the compressed-air tunnel; *No. 1772*, Tests of R.A.F. 34 at negative incidences and of the effect of surface roughness on R.A.F. 34 with split flap in the compressed-air tunnel; *No. 1773*, Note on the design of corners in duct systems. Reports of the U.S. National Advisory Committee for Aeronautics include the following: *No. 585*, Span-load distribution for tapered wings with partial-span flaps; *No. 587* (*Eng. Abs.* **75,** 77), Blower cooling of finned cylinders; *No. 588* (*Eng. Abs.* **75,** 79), Fuel-spray and flame-formation in a compression-ignition engine employing air-flow; *No. 589*, An analysis of lateral stability in power-off flight, with charts for use in design; *No. 590*, Pressure-distribution measurements on an 0–2H airplane in flight; *No. 593*, Cooling of airplane engines at low air-speeds; *No. 595*, Full-scale tests of a new type N.A.C.A. nose-slot cowling. In *Luftfahrt*, **14,** the following articles appear: *p. 168* (*Eng. Abs.* **75,** 143), Airscrews for high-speed aeroplanes; *p. 293*, Determination of the resistance and the air-flow through built-in coolers of various types; *p. 304*, Behaviour of static tubes at high speed; *p. 314*, Theory of submerged aerofoils and skimming surfaces. The two-dimensional hydrodynamical theory of moving aerofoils is dealt with in *Proc. Roy. Soc.*, **161,** *406*. Articles in *J. Roy. Aero. Soc.*, **41,** include: *p. 350*, Aeroplane stability and the automatic pilot; *p. 523*, Plastic materials for aircraft construction; *p. 595*, The formation of ice on aircraft; *p. 609*, Note on the possibility of flight by human power; *p. 635*, Trend of air-cooled aero engines—the next five years; *p. 703*, Critical speeds of monoplanes.

Water-Supply and Sewage-Disposal.

Experiments with fluid friction in roughened pipes, in which the effect of non-homogeneity of the roughness is studied, are described in *Proc. Roy. Soc. Series A.* **161,** *367*. In *U.S. Nat. Bur. Stand. Hydraulic Lab. Bull. Series A. Bull. V-2*, a compilation has been made of current hydraulic laboratory research in the U.S.A. The pressure-losses for fluid flow in curved pipes are studied in *U.S. Nat. Bur. Stand. J. Research*, **18,** *89*. The damage caused to metal by water impact is discussed in *Schw. Bauz.* **109,** *225*, (*Eng. Abs*

75, 68). *Nigeria Public Works Dept. Technical Paper No. 5* describes the method used for the estimation of flood discharges and waterways in the Northern Provinces of Nigeria. The germicidal properties of chlorine compounds for water treatment are discussed in *Iowa Engg. Expt. Station Bull. No. 132.*

The following researches on sewage-disposal have been noted: *J. Sewage Works,* **9,** *207* (*Eng. Abs.* **75,** 154), Activated carbon in sewage-sludge digestion; *p. 224* (*Eng. Abs.* **75,** 151), The oxygen-demand of polluted waters; *p. 285* (*Eng. Abs.* **75,** 155), Coagulation compared with chemical precipitation in the vacuum filtration of sewage-sludge; *J. Western Soc. Engineers,* Vol. **42,** *55,* Activated sludge—recent discoveries.

Lighting, Heating, and Ventilation.

The Department of Scientific and Industrial Research has published *Illumination Research Technical Paper No. 20* on the use of coloured light for motor-car headlights. The interesting conclusion is reached that none of the claims made in favour of using a coloured, and in particular a yellow, headlight beam, has been substantiated. In *J. Inst. Elec. Engineers,* **80,** *636,* optimum symmetrical light-distribution is discussed.

Researches into thermal conductivity include: Measurement of the thermal conductivity of insulating materials, *Zeitschrift für technische Physik,* **17,** *283,* and *Soc. Glass Tech. J.* **21** (*84*), *Abstracts 90*; and a Paper on heat-transmission through walls, giving a description of the new heat-transmission laboratory at the Building Research Station, *J. Inst. Heat. and Vent. Eng.* **5,** *82.*

A study of progress at the Building Research Station in ventilating research is given in *J. Inst. Heat. and Vent. Eng.* **5,** *183.* The nature of noise in ventilating systems and methods for its elimination are discussed in *Heating, Piping,* **9,** *183.* The velocity-distribution in fan-ducts, with particular reference to mine ventilation, is dealt with in *J. South African Inst. Engineers,* **35,** *234.*

Telegraphy and Telephony.

The control of wireless signal variations is discussed in *J. Inst. Elec. Engrs.* **80,** *610*; and on *p. 623* is an article on control of phase-fading in long-distance radio communication.

THE LATE DR. H. H. JEFFCOTT.

SECRETARY OF THE INSTITUTION 1922-1937.

From a photograph by Elliott & Fry, Ltd.

OBITUARY.

HENRY HOMAN JEFFCOTT, B.A., B.A.I., Sc.D., the second son of William Jeffcott, Justice of the Peace, was born in County Donegal on the 6th February, 1877, and died on the 29th June, 1937, at Walton-on Thames, Surrey. From 1895 to 1899 he was educated at Trinity College, Dublin; he then proceeded for 3 years to the Engineering School at Trinity College, Dublin, where he gained high academic honours. From 1902 to 1904 he was trained as an assistant with Messrs. Siemens Brothers and Company, Ltd., at Woolwich and Stafford, and then went to the Openshaw works of Messrs. W. G. Armstrong Whitworth and Company. In 1905 he became head of the Metrology Department of the National Physical Laboratory, London, being occupied with experimental work connected with screw-threads, pipe-flanges, bolt-heads, standardization of engineers' gauges, verification of surveying apparatus for base-line measurement and the design, erection and testing of measuring machines.

In 1910 he was appointed Professor of Engineering in the Royal College of Science for Ireland, Dublin, and held this post until 1922; during the last 8 years of this period he was also Dean of the Faculty of the College. During the War he was responsible for the manufacture of munitions in the College workshops, involving the design and construction of turret lathes and the adaptation of existing lathes by novel methods. He held from 1918 to 1921 the post of Secretary of the Water Power Resources of Ireland Sub-Committee, and was also examiner in Mechanical Engineering in the University of Belfast and in the National University of Ireland. During this period also he was associated with Sir John P. Griffith, Past-President Inst. C.E., in the preparation of a hydro-electric scheme for the river Liffey. In 1922 he was appointed Secretary of the Institution of Civil Engineers, which position he held until his death.

Dr. Jeffcott wrote many papers on engineering and scientific subjects for the Transactions of learned societies upon such matters as whirling of shafts, vibration of beams, elastic deformation, screws and threads, electrical transmission lines, hydro-electric investigations, heat-engines, surveying instruments, etc., and in addition he was the patentee of several inventions, the best-known of which is his direct-reading tacheometer. He always placed

himself with unfailing readiness and courtesy at the disposal of members and of visitors to The Institution, and his death will be keenly felt by all those with whom he came in contact. A memorial service was held at St. Margaret's, Westminster, on Monday, 5 July, which was attended by many members of The Institution and by numerous representatives of other bodies.

He was a Member of the Institution of Civil Engineers, the Institution of Mechanical Engineers, and the Royal Irish Academy. He married Louisa, daughter of George Lang, who survives him and by whom he had one son who died in infancy.

NOTE.

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